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TESTS OF THE CAVITE STEEL FLOATING DRY DOCK.*

By A. C. Cunningham, Civil Engineer, U. S. N.

In the early part of June, 1905, the Maryland Steel Co. reported the Cavite self-docking steel floating dry dock ready for test, having completed the same in one month's less time than the twenty-seven allowed by the contract.

As in the case of the Government floating dock at New Orleans, built by the same company, the Cavite dock was constructed in a shallow basin, and on completion the basin was flooded and the cofferdam at the entrance removed. While flooding the basin the valves of the dock were left open and the dock allowed to fill, so that it continued to rest on the pile and timber foundation on which it was built until the time selected for floating and towing it to the mouth of the Patuxent River, the place selected for test.

When the dock rose from its timber bed it was christened the 'Dewey" after the Admiral of the Navy, by Miss Grace Endicott, the daughter of Rear Admiral M. T. Endicott, Chief of the Bureau of Yards and Docks, Navy Department, under whose administration the dock had been built, and who represented the Navy Department when the dock was floated.

In the preliminary test of the New Orleans floating dock a collier of 6,000 tons displacement was used. The increased confidence in this class of dock for warships, and in the "Dewey" dock in particular, is shown by the fact that one of the latest and heaviest ships, the armored cruiser "Colorado," was selected for the preliminary test.

Before docking the "Colorado," the dock was sunk to 29 ft. draught over the keel blocks and then pumped up light to a freeboard of 2½ ft. The sinking was accomplished in 1 hour and 36 minutes and the pumping up in 1 hour and 2 minutes, showing that the dock is of very rapid action. On this test it was also demonstrated that little or no trimming was required, the dock sinking and rising practically level with all valves open.

The U. S. S. "Colorado" was docked on June 23, 1905, having a displacement of 13,300 tons at that time. The main and docking keel blocks were all set at the same height. In this preliminary test no effort was made to secure speed, and one-half hour was used in making flushing and fire connections. The elapsed time from when the ship landed on the blocks until the keel came out of water was 2 hours and 16 minutes. Pumping was continued until the dock had a uniform freeboard of 2½ ft., only enough excess of water being retained in the side walls and end compartments to give the necessary trim. The "Colorado" was carried on the dock about 24 hours without changing the water ballast. When the dock had reached a freeboard of 2½ ft. with the "Colorado," the deflection on the main keel line in the 500 ft. of length of the dock was about one-quarter of an inch; after about 24 hours the deflection in 500 ft. increased to about 1 3/16 ins. After undocking the "Colorado" the dock was found to have practically straightened without retaining any set.

After the undocking of the "Colorado," deflection observations were continued for three days, and variations in deflections with the dock unloaded, due to temperature changes, of seven-eighths of an inch were noted.

The battleship "Iowa" was docked on June 27, 1905, for a record test, having a displacement of 11,600 tons at the time, and was carried on the dock for 48 hours. The specifications required that a 16,000-ton ship should be raised in four hours from the time the ship took the blocks until the keel was out of water. For the equivalent of a 16,000-ton ship the dock was pumped to a freeboard of 4½ ft. From the time the "Iowa" took the blocks until the keel was out of water was 1 hour and 37 minutes; to the time the dock had a freeboard of 4½ ft., 2 hours and 42 minutes. During the docking of the "Iowa" one of the three pumping engines was out of commission for 42 minutes with a slipped eccentric, so that the actual time of operation of the dock is about half that allowed by the specification.

The "Iowa" was docked by uniform pumping, as in the case of the "Colorado," and carried for 48 hours without change of water ballast in the dock. The specification required that when a ship had been docked by uniform pumping until the dock had a freeboard of 2 ft. the deflection in the entire 500 ft. of length of the dock should not exceed 3 ins. When the dock reached a freeboard of 4½ ft. with the "Iowa," the deflection was about 2 ins. During the first 24 hours, the dock remaining uniformly pumped, the deflection increased to 4 ins. in the 500 ft., and during the second 24 hours showed a recovery to 3⅜ ins.

Immediately following the undocking of the "Iowa" the dock was pumped up to the same depth of water in the compartments as when the ship was docked, which gave a freeboard of 9 ft. 6 ins., and it was found that the dock had a hog of 1 in. During the night this hog disappeared, and early the next morning was a ½-in. sag. The greatest deflection in the bearing length of the "Iowa" while carried on the dock was about 1¾ ins. The deflection observations indicate that there was no permanent set caused by the docking, and that temperature variations may cause considerable hog or sag.

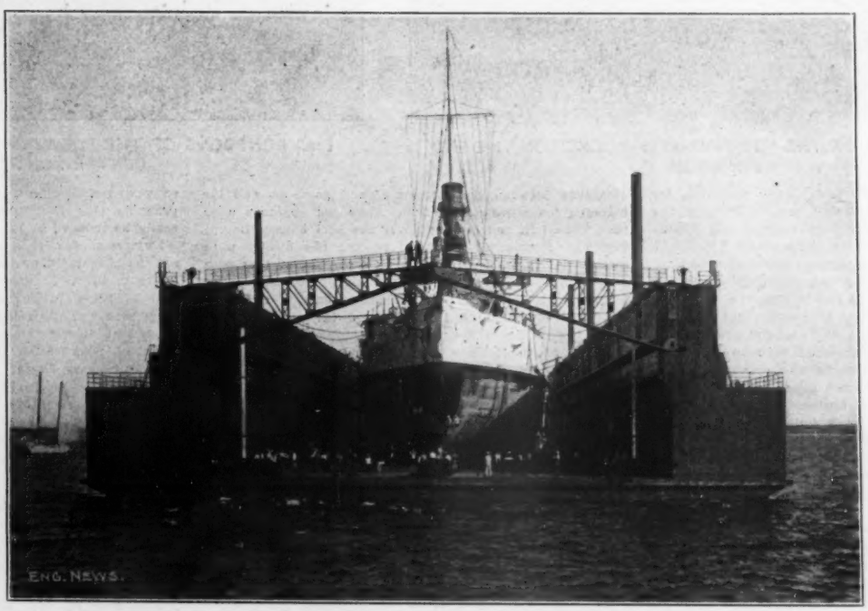
After the undocking of the "Colorado" the main and docking keel blocks were found to be uniformly indented about 1/16 in. with no crushing. No change was made in the blocks for the "Iowa," and after undocking she was found to have rested even more easily than the "Colorado."

With 2 ft. of freeboard and 1 ft. of water remaining in the pontoons the carrying capacity of the dock is 18,500 tons, and a 20,000-ton ship could readily be docked with sufficient freeboard to admit of easily working on its bottom.

The results of the docking tests with the "Dewey" dock show that it is considerably in excess of the contract requirements in strength, time of operation and capacity, and is in all respects a very noticeable advance on all floating dry docks which have been so far projected or built. The greatest innovation is the requirement of uniform pumping. This not only insures safety from careless or unskilled handling, but makes it possible to dock nearly all ships with keel straight, or with as much hog or sag as circumstances may render desirable, by suitably distributing the water ballast in the dock. The ship may also be hogged or sagged while on the dock, should occasion arise for so doing. With 30 ft. of water over 4-ft. keel blocks the side walls of the dock have a freeboard of 11 ft. By taking the keel blocks down to 2 ft. and sinking until the side walls have a freeboard of 3 ft., 40 ft. of water may be had over the blocks, so that a ship may be taken into the dock in any condition of disablement.

Although a heavier ship, the deflections with the "Colorado" were less than with the "Iowa," on account of the much longer bearing length of the "Colorado's" keel. With the uniform pumping the strains caused by the "Iowa" were much greater than will be caused by any other ship in the service, projected or built; so that the dock has had the most severe test that can be imposed on it.

The dock was constructed under the supervision of Civil Engineer Leonard M. Cox, U. S. N., and tested by a board of naval officers, consisting of Captain Adolph Marix, senior member; Naval Constructor J. H. Linnard, Commander J. F. Parker, Commander W. F. Worthington, Naval Constructor J. H. Rock, Civil Engineer A. C. Cunningham and Assistant Civil Engineer J. S. Shultz. The board was assisted in the tests by Civil Engineer Cox.



THE U. S. FLOATING DRY-DOCK "DEWEY," WITH THE BATTLESHIP "IOWA" COMPLETELY DOCKED.

*From the "Journal of the American Society of Naval Engineers.

THE SELF-DOCKING TESTS OF THE STEEL FLOATING DRY DOCK FOR CAVITE, P. I.*

By Leonard M. Cox, Civil Engineer, U. S. N.

In the specifications forming a part of the contract between the Government and the Maryland Steel Co. for the construction of the steel floating dry dock for Cavite, P. I., it is provided that the structure, after completion, shall be given a thorough working test before final acceptance. This test, as provided for in the contract and its specifications, comprises three distinct operations: The lifting, within a given time, to a specified freeboard and with limited deflections, of the heaviest type of cruiser; the lifting, under similarly specified conditions, of the heaviest battleship; and, finally, the lifting of its own hull clear of water.

In regard to the desired self-docking qualities of the dock, the specifications provide that it "shall be so designed and arranged as to be readily self-docked without the aid of divers or auxiliary constructions;" that the working stress of any portion of the dock or its connections shall be limited to "15,000 lbs. per sq. in. in self-docking, with a wind pressure of 30 lbs. per sq. ft. of exposed surface;" that "when self-docked all underwater portions shall be raised to a height of not less than 5 ft. and shall be safely and readily accessible for inspection, painting and repairs;" and, that with the dock at light-draught line, "all self-docking and strain-transmitting connections shall be above water."

The dock is built in three sections. The center section, or pontoon, is 316 ft. in length, with side walls overhanging 80 ft. on either end. Two end pontoons, each 90 ft. in length, with low independent side walls, are attached to the center pontoon in such a way that the overhanging side walls of the latter are enclosed between the independent walls of the end pontoons and rest directly on their decks when the sections are connected for docking ships. There are vertical and horizontal connections between the pontoons, the vertical connections consisting of seven elements, each made up of 44 2-in. bolts; the horizontal, of four elements, one along each edge of the overhanging side walls, each made up of 96 1½-in. bolts. The lower ends of the vertical connections are out of water at light draught.

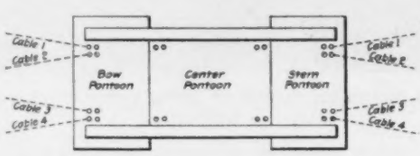
The self-docking is accomplished in two stages. After removing the connection bolts, the end pontoons are hauled clear of the overhanging walls of the center pontoon, turned so that the direction of their lengths is perpendicular to the axis of the dock, and brought over the deck of the center pontoon and centered on blocking, as in the docking of ships. The center pontoon is then pumped to the desired freeboard by the main pumping engines. The second stage of the operations has for its object the lifting of the center pontoon. The end pontoons are submerged to a depth giving 12 ins. clearance between the blocks and the bottom of the center pontoon at light draught, and are then drawn under either end and centered, after which they are pumped to the desired freeboard by means of a separate pumping plant installed in the independent side walls, to which steam is furnished by a flexible hose led from the forward and after boilers.

On Saturday, July 15, 1905, the work of arranging the blocks of the center pontoon for receiving the two ends was finished, and all vertical connection bolts, together with half of the horizontal bolts, were removed. On Monday, cables 1 and 2 (see accompanying sketch) were slipped from the end pontoon bitts, starboard side, forward, and made fast by hawsers to bitts on the top of the overhanging side walls. On Tuesday, cables 3 and 4, forward, were similarly slipped and belayed on the port side-wall bitts. All horizontal connections were removed and steel drift pins placed in the bolt holes at intervals of about 25 ft., with men standing by for each pin. At 12.15 p. m. the flood valves of the forward pontoon were opened, and ten minutes later the horizontal connection diaphragms came gently apart. With the aid of a small tug boat, the end pontoon was hauled clear of the overhanging side walls at 12.53 p. m., and made fast against the starboard side of the center pontoon at 1.35 p. m. Cables 2 and 3 were then slipped from the tower-deck bitts and made fast to the forward-deck bitts of the center pontoon with two shots of chain which had previously been detached from cables 1 and 4. An anchor-hoy and tug was used in shifting moorings and handling cables.

On July 19, cables were slipped from the after pontoon bitts and made fast to the side-wall bitts as was done in the case of the bow pontoon. All horizontal connection bolts were removed and the flood valves opened at 10.20 a. m. The pontoon began to drift out with the tide at 10.49, and was hauled to the port side of the dock and made fast at 12 m. Cables 2 and 3 were then slipped from the tower bitts and made fast to the after center pontoon bitts as for the bow pontoon.

On July 20, the center pontoon was submerged to an average draught of 33 ft., giving a little over 12 ins. clearance between the bottoms of end pontoons and the blocking for a draught of 9 ft. The end pontoons were then hauled in and centered over the keel blocks. Pumping was commenced at 12.26 p. m., the blocks were awash at 1.25, and the pumps were stopped at 1.40 with a freeboard of 18 ins., the total time of pumping being 1 hour and 20 minutes. All mud valves in end pontoons were opened at 1 p. m., and the compartments drained with the falling water.

During the next three days the pontoons were held on the blocks and the time was spent in cleaning and repainting such parts of their bottoms as seemed to require it, and in arranging the blocking on the ends for seating the center pontoon. On July 24 the mud valves of the end pontoons were closed at 8.40 a. m., the flood valves were opened at 9.15 and closed with the pontoons afloat at 10 a. m. The pontoons were lashed together in tandem and drawn out with the tide, being clear of the dock at 11.31 a. m. Both pontoons were then made

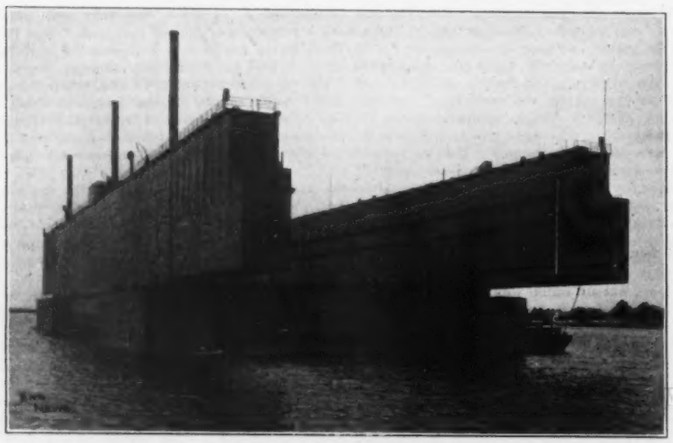


fast to the starboard side of the center pontoon. When floated, the end pontoons had an average draught of 4.59 ft., making their joint displacement 3,164 tons.

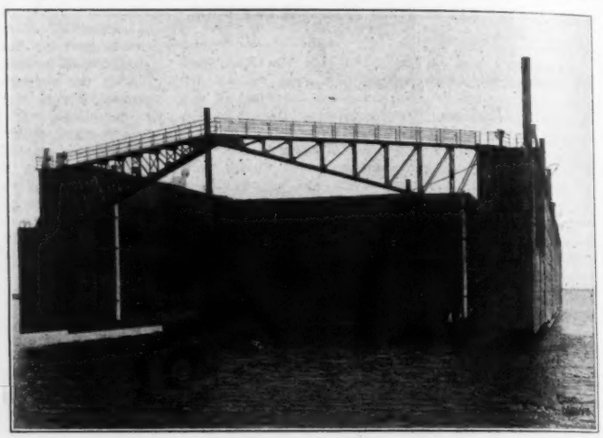
The 25th of July was consumed in arranging the blocking, and on the 26th the work of docking the center pontoon was begun. Cables 2 and 3 were slipped and led by hawsers to side-wall bitts, the dock being held in the meanwhile by cables 1 and 4. At 8.30 a. m. the stern pontoon was sunk to sufficient depth to permit its being hauled under the overhangs, and at 11.40 it was in position and ready for sinking to full draught. At 1.05 the flood valves were opened, and at 3.50 p. m. the pontoon had reached an average draught of 31 ft., the sinking being carried on very deliberately on account of the possible effect of the strong ebb tide, nearly always to be contended with at this point. After reaching this depth the pontoon was drawn under the center pontoon and centered over the blocks. The pumps were started and continued until the pontoon reached a draught of 28 ft., and the center was firmly seated.

On July 27th the dock was simply held in position and observations made for possible settlement. On July 28 the bow pontoon was drawn into position, sunk to a draught of 31 ft., drawn under the center pontoon and, at 2.20 p. m., pumped to a uniform draught with the stern pontoon. Both end pontoons began pumping full speed at 2.20 p. m. and reached a freeboard of 18 ins. at 7 p. m. After holding the dock in this position for four days, the work of undocking was commenced, but, on account of bad weather, this work was discontinued and the end pontoons were held at 26 ft. draught until August 2, when the stern pontoon was hauled clear and under the center and connected in its proper position. Two more days were lost on account of the weather, and the bow pontoon was withdrawn and connected on August 4. No difficulty was encountered in entering the connection bolts, and each pontoon was sunk, hauled out, pumped to proper draught and connected in 12 hours' time.

The total working time consumed in the self-docking tests, exclusive of the time spent on the blocks, is about 15 days, and, when it is taken into consideration that this work is absolutely new, that it was necessary to proceed with caution during the first test, and that the crew was naturally unused to its work, it would seem probable that the time for future self-dockings could easily be reduced to 10 or 11 working days.



CENTER PONTOON OF THE "DEWEY" SELF-DOCKED ON THE END PONTOON.



END PONTOONS OF THE "DEWEY" SELF-DOCKED ON THE CENTER PONTOON.

Numerous devices have been exploited in the past, all claiming more or less novel means to enable floating docks to lift their own hulls for inspection and repairs. Few of these devices appear practical, and still fewer have been actually tried on large structures. The most prominent among the self-docking types is the Clark & Stanfield modification of the Rennie type, represented in this country by the naval docks at Algiers, La., and at Pensacola, Fla. The Algiers dock was successfully self-docked by her builders during the acceptance tests, and required for the operation about 40 days. It succeeded in lifting the pontoons to a height of 4 ft. and the side walls to a height of 21 ins. above water. The Pensacola dock has docked her intermediate pontoons, but is only now, for the first time, attempting to lift the end sections. So far as I am aware, the new Bermuda dock has never been completely docked, nor has the new naval dock at Pola, Austria, the latter being an entirely new design.

The Cavite dock has, so far, proven a complete success, and the self-docking features are particularly satisfactory. In the light of experience with other self-docking types, it would seem that, as regards the saving effected in time and labor, facility of control and simplicity of operation, the self-docking problem has been satisfactorily solved and the greatest objection to the floating dock per se completely eliminated.

The self-docking tests were conducted by a board of which Captain Adolph Marix was senior member, and Naval Constructor W. G. Du Bose and Civil Engineer Leonard M. Cox, members.

Much interest has been felt in the plans for transporting this great floating dock to Manila, a voyage of 14,000 miles. It will be apparent that the resistance of the structure to movement through the water is so great that it can only be towed at a very slow rate. The complete dock is 500 ft. in length and 134 ft. wide, and its total height is 63½ ft.

The dock is now at Solomon's Island, and preparations for its departure are actively going on. The command of the expedition, which will include besides the dock the two colliers, "Brutus" and "Caesar," and the refrigerator ship "Glacier" as tender, will be in the hands of Captain William F. Fullam, U. S. N. Captain James Wood, master of tugs at the Norfolk Navy Yard, will

*From the "Journal of the American Society of Naval Engineers."

charge of the dock itself during the trip. It is expected that the dock will leave this country in about three weeks, but the time of its reaching the Philippines is of course problematic, depending greatly upon the weather. It will be towed from Chesapeake Bay to the Philippine Islands by way of the Suez Canal, the route around Cape Horn being considered too dangerous at this time of the year for such an undertaking. It is estimated that a speed of about 100 miles a day can be attained with the dock in smooth weather.

In preparation for the voyage, the dock is now being equipped with four 9,000-lb. anchors, two at each end, operated by a capstan in the center of the dock. There will be two towing bridles at each end, handled by a temporary bridge across the after end, and a tow line 1,000 ft. long is to be used attached to it. The dock will be equipped with search lights, wireless telegraph, extra coal bunkers, and extra supply of water in two of the regular water-tight compartments. It will have a full line of stores and equipment. Besides the regular dock crew, there will be 20 enlisted men to handle moorings and tow lines. The colliers "Brutus" and "Caesar" have been detailed to tow the dock, with the refrigerator ship "Glacier" as tender to take hold of the stern if necessary to swing the dock around.

Not much difficulty is anticipated from rough weather, for the dock can if necessity requires be sunk so that only the side walls will project above the water and be exposed to the elements.

A NEW METHOD OF PUMPING SAND BY MEANS OF COMPRESSED AIR AT THE PLANT OF THE UNITED STATES SILICA CO.

By Lucius I. Wightman.*

One of the most unique applications of compressed air is found at the plant of the United States Silica Co., Ottawa, Ill., about 80 miles from Chicago. At this point is an immense deposit of silica sand, very fine, almost snow white, of the best grade for glassmaking purposes. The property of this company at this place has been worked for some years and at present the excavation is about 800 ft. long, 600 ft. wide, and with a maximum depth of about 100 ft.

The cost of removing this sand and hoisting it to the plant on the surface has been steadily increasing with the depth reached. Recently a point was reached at which a shut-down of the property seemed imminent unless some more economical method of lifting the sand could be devised. In this crisis a contract was placed with the Pneumatic Engineering Co., a department of the Ingersoll-Rand Co. of New York, for the installation of a Harris compound displacement pump especially adapted for this particular work. Briefly, this system involves a direct displacement of the contents of a tank by air under pressure, this air being in turn returned to the compressor for recompression. Advantage is thus taken of the expansive force of the air—a feature distinguishing the Harris system from other pneumatic displacement pumps.

*838 Carlton Ave., Plainfield, N. J.

In the Ottawa plant a sump was dug in the lowest part of the sand pit in which a Harris pump was installed, consisting of two steel tanks 5 ft. in diameter and 5 ft. in height. Upon these are mounted the connecting pipes and valves by which the tanks are filled and discharged alternately. The double air line of 3½-in. pipe leads to the automatic switch and compressor in the engine room on the brink of the pit. The compressor is a Rand Class C, straight-line, steam-driven machine, running under 80 lbs. steam pressure and delivering air at a pressure of 80 lbs. at the maximum of the pumping cycle. The discharge pipe from the pump to the sand bin is a 7-in. pipe and the total distance of delivery is about 400 ft., with a vertical lift of 110 ft.

The method of excavating the sand is at present as follows: A constant flow of water, which enters the pit on all sides from springs, is collected in a sump which is the source of supply for a small duplex steam pump which takes the water and discharges it through a hose and nozzle. The stream is played on the walls of the pit washing the sand down, and the mixture of sand and water is directed into a box beside the main pump. The pump suction pipe enters this box and the partial vacuum created in the tanks draws the sand and water into the tanks, from which it is discharged by compressed air in the next cycle through the delivery pipe to the bin above. The sand mixture varies in quality from one of 5% of sand to one of 65% of sand by weight. The average performance of the pump is 50 tons of sand alone raised per hour, or 500 tons of sand per 10-hour day. The work done by the pump will be further appreciated when it is considered that the supply of water entering from the springs is from 300 to 500 gallons per minute, all of which is discharged with the sand through the pump.

It might naturally be expected that the rapid pumping of such an abrasive material as sand would soon destroy the pump mechanism, but after an operation of several months there has been found to be no appreciable wear on this system. On top of each tank there is a check valve preventing the return of the material in the discharge pipe. This is of the simplest form, consisting of a rubber covered ball resting on a ring seat, and inspection has shown that while the seats have worn, they have worn uniformly and present the appearance of a highly polished turned ring. The balls themselves show no wear whatever. The pressure controlling valves are located in the engine room above and at no time are in contact with the material pumped.

This system works continuously; during the day it pumps sand and water in mixture; during the night it pumps water alone, keeping the pit empty and ready for resumption of operations on the following day. It is found that the maximum volume of water entering the pit can be handled when the pump tanks are filled and emptied at the rate of about one a minute.

The introduction of this method of pumping simplified the problem of removing the sand in many ways. Its operation calls for three men; the engineer looking after the compressor, who also cares for other machinery in the surface plant; the pump attendant, whose duty it is to see that the flow of sand and water is properly directed to the gathering box; and the hose man who handles the stream of water for removing the sand from the walls.

The sand mixture is discharged from the pump into a receiving bin at the surface where it is allowed to settle; the water flows off through a wasteway. This partially dried sand is then removed from the bins and put through the usual process of drying and separating. The operation of the surface plant offers no unique features; but the introduction of this pneumatic system of pumping in the pit has been the solution of a serious problem which was confronting all the sand producers of that section. The illustration shows the interior of the pit with the pump in position and the pipe lines leading to the plant on the brink. A very good general idea of the difficulties encountered and the methods employed may be had from an examination of this view.

The system which the new method replaces was, briefly, as follows; the work was carried on in three levels; the sand was washed by the hose to the lowest level, or sump, where it was hoisted by a vacuum steam pump to the second level. Here a single outside-packed direct plunger pump lifted it to the third level, from which it was elevated by a scraper conveyor to the bins. The seepage water was handled at night by a duplex steam pump. All of these pumps were supplied from the surface boiler plant through long steam lines; and the steam and fuel consumption was enormous, due both to the excessive pipe condensation and to the wasteful non-expansive features of the pumps. Furthermore, the whole system was liable to freeze up solid in winter. The depreciation on the machinery involved was very heavy, the plunger pump especially wearing rapidly under the abrasive action of the sand mixture and losing what little efficiency it had. Fuel and maintenance costs were enormous.

With the pneumatic sand lift, the only long steam line is the small one to the hose pump. The rest of the system, being under pressure, is proof against freezing, and its freedom from abrasion has already been noted.

VIEW OF SAND PIT OF UNITED STATES SILICA CO., OTTAWA, ILL., SHOWING COMPOUND DISPLACEMENT PUMP FOR PUMPING SAND.

A THROUGH PULLMAN TRAIN TO THE CITY OF Mexico is to begin running on Jan. 16 from St. Louis via the Iron Mountain line and the National R. R. of Mexico. The schedule time is to be 55 hours from St. Louis to Mexico City and a through dining car will be run, which is said to be the first regular dining car service to be established in Mexico.

A VERY LONG BRIDGE is being built over the Hoang-Ho, China, for the crossing of the Hankau-Peking Railway. The river is about two miles wide at the crossing, and the bridge will have 102 spans aggregating about 10,800 ft. A recent consular report which gives some information about the railway says: "The bridge is not constructed of heavy structural steel, but is of more frail design, which, however, will apparently prove of ample strength for the light locomotives and cars in use on this road." The report is more intelligible in its statements about the foundation work, which seems to have been very difficult, as the bottom is quicksand in many places, and the river has a rapid current.

It was found to be practically impossible to sink ordinary piling, and finally screw piling was successfully used. Although some of the piling has been sunk by machinery 40 ft. into the bed of the river, there is said to be a possibility of a sudden rise in the river carrying away some of the piers. This is largely conjecture. It has been planned to drive the piling down to a point beneath the scour of the river. No caissons were used, and on account of the great width and strength of the stream, the fact that there is no solid rock foundation, that the main channel is 33 ft. in depth and that in flood periods this depth may be increased to over 45 ft., has occasioned much conjecture as to the result of the bridging undertaking.

The bridge should be completed within a very short time. The trip between Hankau and Peking (754 miles) is expected to be reduced to 36 hours when the bridge is put in service.

DESIGNS FOR TERMINALS FOR THE MANHATTAN BRIDGE, NEW YORK CITY.

Past experience in New York City in the transportation of persons between the boroughs of Manhattan and Brooklyn by bridge has established very definitely that in any given case the bridge structure itself is only part of the general problem to be solved. A matter fully as important is to provide the bridge with suitable terminals and means of approach. So soon as it becomes technically and commercially feasible to construct a bridge of any kind at a certain location on the East River, it is just as feasible to make the bridge of sufficient capacity to handle any specified amount of traffic. But whether the bridge can ever be made to handle such a traffic, or indeed even half that traffic, depends essentially upon the facilities provided for bringing traffic to and from the bridge.

At the time the first bridge between the (then) cities of New York and Brooklyn was built—the Brooklyn Bridge—this fact was quite unrecognized. The cities have been reaping the reward of this early ignorance ever since, in the form of throttled traffic and intolerable crowding at the bridge terminals. Twenty-five years later, when the second East River Bridge, the Williamsburgh Bridge, was designed, the error was made of assuming that the detail design of terminals and their construction could be carried on at leisure after the important thing, the bridge itself, was completed. As a result, during the twenty months since the bridge was opened for traffic it has served its purpose only in very imperfect manner, and its full utilization still awaits the final design and construction of terminal and connecting transit facilities.

The Manhattan Bridge, soon to be put under construction, will benefit by the experience with the older bridges. Although the superstructure has not yet been contracted for, complete designs have been made for terminals and connections at the Manhattan and Brooklyn ends, so that it will be possible to realize full service from the bridge as soon as the structure is completed. These designs are exhibited in the five illustrations shown herewith.

The fundamental principle which was evolved out of the Brooklyn Bridge terminal problem is that terminals must be avoided altogether, or as much as possible. In order to enable the bridge itself to be utilized to fullest capacity, the terminals would have to be very large and elaborate; they would need to be double at each end, comprising a terminal for the lines of railway crossing the bridge, and a terminal for the city lines leading to the bridge. Now, of the mass of passengers crossing the bridge, only vanishingly few start from or are bound for the end-point of the bridge, or its immediate neighborhood; the vast majority starts from or desires to reach points on the urban transit lines leading to or from the bridge. It follows that the needs of the traffic are best served by combining the urban lines and the bridge lines in through service, thereby avoiding the delay and inconvenience of interchange at the terminals. This arrangement, moreover, renders unnecessary the large double terminals otherwise essential. In the case of the East River bridges it is not practicable at the present day to apply this principle fully, but subject to certain restricting conditions it is carried out as far as possible in the present designs for the Manhattan Bridge terminals.

The main interfering factor is the fact that the present urban transit systems on the two sides of the river are radically distinct. The Brooklyn surface (overhead trolley) and elevated lines form a single system, the Manhattan surface cars (conduit trolley) are a single system, and the Manhattan elevated lines are another system. The physical equipment, as well as the operating conditions of the Manhattan and the Brooklyn systems are quite different, a[bove] all there is a strong antagonism betwee[n] two groups of lines, which would act [ac]tively against either interchange of traffic o[r] mutual invasion of the other's territory. In co[n]sequence it was not possible to adopt a complete "[th]rough" system of operation, but it became n[ec]essary to provide a terminal for Manhattan tran[sit] at the Brooklyn end, and for Brooklyn transit at the Manhattan end.

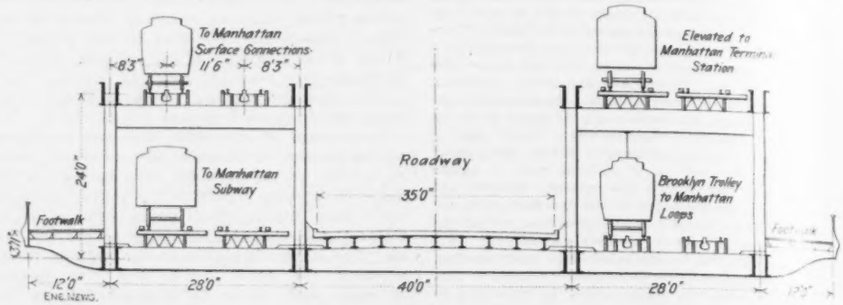


FIG. 1. CROSS-SECTION OF THE MANHATTAN BRIDGE ACROSS THE EAST RIVER, NEW YORK CITY, SHOWING ARRANGEMENT OF TRACKS, ROADWAY AND FOOTWALKS.

The earliest designs for a bridge across the East River at the site of the Manhattan Bridge provided for a central and unobstructed roadway in the middle of the bridge and in continuation of an extension of Flatbush Av. (Brooklyn), and this has been persistently adhered to in all subsequent plans for the bridge. The width of the bridge was made 120 ft., including: A central roadway 35 ft. in width; two spaces (one on each side of this roadway) for railway tracks, these tracks to be placed in two stories, one above the other; and two sidewalks for pedestrians, one on each side of the bridge.

It was originally intended to use the two tracks on the lower or roadway floor for trolley cars alone, and the two tracks on the upper floor for elevated trains alone. It has since developed that the elevated tracks may not be extended beyond the terminals in Manhattan, and a subway route has been laid out by the Rapid Transit Railroad Commission through the Flatbush Av.

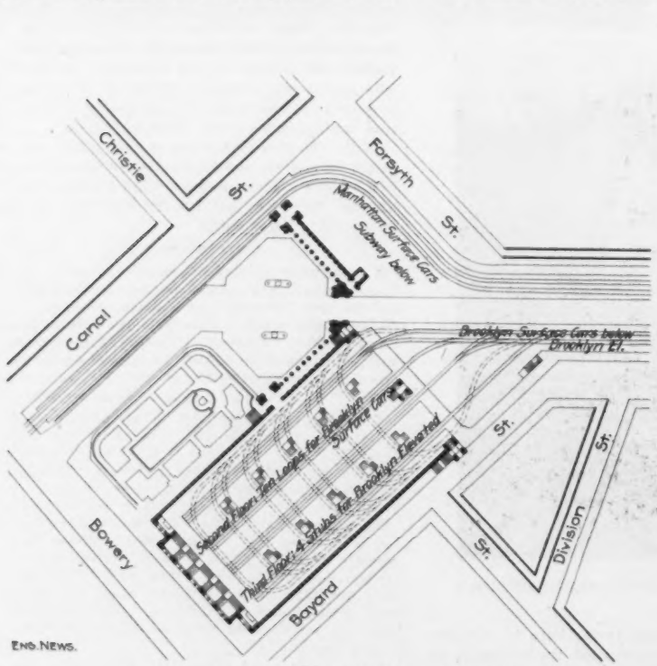


FIG. 2. PLAN OF MANHATTAN TERMINAL.

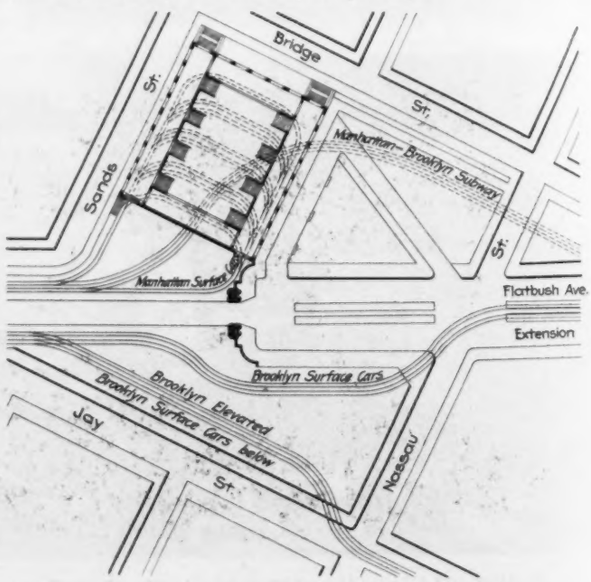


FIG. 3. PLAN OF BROOKLYN TERMINAL.

FIGS. 2 AND 3. PLAN OF TERMINALS OF MANHATTAN BRIDGE, NEW YORK CITY.

O. F. Nichols, M. Am. Soc. C. E., Chief Engineer Department of Bridges. R. S. Buck, M. Am. Soc. C. E., Chief Engineer of Manhattan Bridge.

Carrere & Hastings, Consulting Architects.

sion, over the Manhattan Bridge and through Canal St. in Manhattan. The subway routes can only be carried on the bridge with a grade of 5% at the terminals, and these routes on the bridge must occupy a portion of the lower

FIG. 4. PERSPECTIVE VIEW OF MANHATTAN TERMINAL OF MANHATTAN BRIDGE.

floor on the bridge since the grades to reach the upper floor would be prohibitory.

The space on the bridge has therefore been divided as follows: 1. A central roadway 35 ft. in width; 2. Two footwalks, each 12 ft. in width, on the north and south sides of the bridge; 3. A double track subway route on the north tracks of the main floor for the Manhattan Bridge route of the Rapid Transit Commission; 4. A double track trolley line on the south tracks of the main floor for Brooklyn trolley cars running to the Manhattan terminal; 5. A double track trolley line on the north tracks of the upper floor for Manhattan trolley cars running to the Brooklyn terminal; and 6, A double track elevated railroad line on the south tracks of the upper floor for Brooklyn elevated railroad trains running to the Manhattan terminal. The cross section shown as Fig. 1 herewith exhibits this apportionment of space.

As appears from the preceding statement of transit lines on the bridge, two sets of lines terminate at the Manhattan end of the bridge (a double-track surface car line and a double-track elevated line, both of the Brooklyn system), and one line terminates at the Brooklyn end (a double-track surface car line of the Manhattan system). Terminal buildings or headhouses are provided at the two ends of the bridge, containing facilities for terminal traffic interchange of these lines.

The design of the terminals includes, besides, the arrangement of railway lines and terminal buildings in such a way as to leave the central roadway unobstructed, and to give suitable connections between the through lines on the bridge and their street continuations.

Figs. 2 and 3 show, respectively, the Manhattan and the Brooklyn terminals in ground plan. Both terminals embody the idea of a terminal plaza accessible by two or more main streets, an open roadway through the center, a terminal house to one side, and other transit lines passing off along the sides without obstructing or crossing the roadway. The plazas and their buildings are partially enclosed by masonry walls, and with the buildings are so worked out as to make each a harmonious architectural entity.

Pedestrian traffic will in the main be taken care of by connections from the two footwalks to lateral streets.

The terminal building on the Manhattan side, which fronts on the Bowery, is a three-story masonry structure with steel floor-framing. The ground floor, on the street level, forms a concourse. On the second floor are ten terminal loops for Brooklyn trolley cars, as shown by dotted lines in Fig. 2. The platforms between the loops are reached by stairs from the concourse. On the third floor will be at least four terminal stubs (and five platforms) for Brooklyn elevated trains.

The Brooklyn terminal building, fronting on Bridge and Sands Sts., will furnish five terminal loops for surface cars from Manhattan, on the second floor, and will have a ground-floor concourse like that of the Manhattan structure.

The Brooklyn trolley lines approach the bridge through either Jay St. or Flatbush Av. extension, or both, going to the south roadway of the main (lower) floor of the bridge. Elevated trains from Brooklyn will pass through Jay St. or across private property onto the bridge terminal and thence to the south roadway of the upper floor.

FIG. 5. PERSPECTIVE VIEW OF BROOKLYN TERMINAL OF MANHATTAN BRIDGE.

Manhattan surface cars may pass through Canal St. or the Bowery, passing by an incline alongside the central roadway to the north side of the upper floor. Underneath these tracks will be the new subway, running on the north tracks of the lower floor, and passing off the bridge into subways at both the Manhattan and the Brooklyn ends.

The entire terminals have been worked out with constant regard to suitable architectural treatment, in order that they may be ornamental as well as convenient terminals to the bridge. The perspective views of the two terminals reproduced in Figs. 4 and 5 herewith give a good idea of the effect attained. The architectural part of the design was done by the firm of Carrere & Hastings, of New York City. The planning of the terminals as a whole was carried out by the Department of Bridges of New York City, Mr. O. F. Nichols, M. Am. Soc. C. E., Chief Engineer. Mr. R. S. Buck, M. Am. Soc. C. E., as Chief Engineer of the Manhattan Bridge, was in direct charge of the work.

A REVIEW OF THE ENGINEERING PROPOSALS FOR THE PANAMA CANAL.

PART II.

By Wm. Ham. Hall, M. Am. Soc. C. E.

(Part I. of Mr. Hall's paper, published in our last issue, was intended to be exactly what it was called, "A Statement of Panama Canal Engineering Conflicts." Except where it seemed necessary to make clear exactly what the several proposals have been and how they have conflicted or appeared to conflict with each other, Mr. Hall states that he endeavored not to admit anything like comment. Part II. is a review of those several proposals, and, as such, necessarily, a criticism, though not a captious one.—Ed.)

The engineering conflicts as to what to build for a Panama Canal, are, to say the least, unfortunate alike for the public and for all engineers. Professional timidity, personal or professional friendships too often, in our United States at least, stand in the way of good general understanding being had of such subjects. Our American Society of Civil Engineers took this Isthmian Canal subject up twenty-five years ago on the basis of a paper then presented, and there was a good discussion of it, not only before the society, but before the public. Then but little was known of the important facts, and the project was a foreign enterprise. Now much more is known of the controlling physical factors, and the undertaking is American. In this view, it is to be regretted that American engineers have not been more outspoken upon it in public during the past several years. Mr. George S. Morison's two papers read before the American Society of Civil Engineers, 1902-03, and the discussions thereon, seem to have been forgotten in the short time to the present. It is a mistake to stand back for fear of disagreeing with some one in authority or official position. No engineer, no matter what his position, should feel that he is above criticism. The really great engineer may look upon his frank and open critic as his best professional friend. If there were more frank, open and polite professional plain speaking among engineers, there would be less covert undermining of each other, and the profession would be the gainer in self-respect and public esteem.

MEANING OF MEAN-SEA-LEVEL IDEA.—It is to be understood that I advance herein no strenuous opinion on any point except one, and this is, that a mean-sea-level canal should not be built across the Isthmus of Panama. It may be that it will be best to hold the water level as low as the plane of ordinary high flood tide in the Pacific, which is between six and nine feet above mean sea level, and carry that surface through as a summit level. But while I feel that my mind is fully open to conviction for this high-tide level, as compared to any higher, on the marshaling of facts and estimates and upon hearing the views of other engineers, I fear I should be very firm against cutting the canal to such a depth, namely, a mean sea level, that vessels would have to be lowered into it at the Pacific end, nearly half the time. That, it seems to me, would be a wasteful engineering error—to cut a waterway from sea to sea, through a summit 333 ft. in elevation, into which nearly one-fourth of all the vessel entrances would have to be effected by lowering them, and nearly a fourth of the exits, by raising them. And for what? To save one lock and one lockage near the Atlantic end; for, remember, there must be a lock near the Pacific end on account of the tidal movement, and that lock could as well raise and deliver all vessels to the high-tide plane, as raise or lower them to the mean tide plane.

I must confess at the outset, therefore, that the proposal to buy the advantage of floating ships from the Atlantic into the canal without lockage at all, at the expense of arranging to lower them into it almost half the time from the Pacific, does not appeal to me as being sound, especially when the cost will be the deepening of the canal six to nine feet for its entire length, and the saving will be only one lock for about a 7 to 10-ft. lift. (A foot of these latter figures is on account of the tide at Colon.)

That six to nine feet of excavation would necessarily be the most expensive six to nine feet in depth of the entire cutting. Aside from its being the deepest below water level, and probably the most expensive per cubic yard, it, of course, would lower the whole cutting, slopes and all, that much, and would extend the slopes out further and make more excavation on them than any six to nine feet above it.

It is a noble sentiment—"The joining of the Oceans!" But in this case they cannot be joined, without a gate between them. Nature, in the Pacific tidal movement, forbids. All engineers and even M. de Lesseps, the optimistic promoter, bowed to that. Then, what is the next best thing that can be done? Is it to design the work so that we will lower vessels into it by a lock at the Pacific end for nearly one-half of the time? or so that we will raise them into it at the Atlantic end, to Pacific full tide level, all of the time?

A WATERWAY IN A HOLE.—This is the question which it seems must unavoidably confront the advocates of a sea-level waterway. Thus far all of the sea-level projects have been for the mean-sea or say, mean-tide plane. The extreme tide movement is about two feet at the Atlantic and twenty feet at the Pacific end. Suppose there were twenty feet of tide at the Atlantic as well as the Pacific terminus. Would you excavate for a mean-tide level then? Think a moment. A lock would be an absolute necessity at each end. For about half the time your water plane would be below sea-level at both ends, or for more than half the time at one end or the other. With respect to the waters you sought to join, for half the time your canal would be down in a hole. It would cost much money for the extra excavation to put it there. Would it seem rational to do it? The locks could as well perform the function of raising all the entering vessels to the high tide plane.

I hope the thoughtful engineer, even though he has been inclined to the sea-level idea for the Panama Canal, will agree with me in this—that if there were twenty feet, say, of tidal movement in both seas, instead of in one only, it would be an absurdity to put the canal water plane at any lower than about ordinarily high full tide. If, then, it would be absurd to stick the canal down in a hole with respect to the water as it would be half of the time at both ends, how are we to look upon the proposal to put it in a hole with respect to the water plane half the time at one end? It would not be polite to speak of this latter proposition as any part of an absurdity, though logically, it would seem to be half of an absurdity. We might, however, refer to it as a strange oversight.

ANALYSIS OF OBJECTIONS TO LOCK-LEVEL CANAL.—The alleged objections to a locked canal are: (1) Limitation of size of shipping by reason of unalterable lock dimensions. (2) Inconvenience in enlarging the waterway, even if locks are from the beginning made large enough. (3) Loss of time in lockage. (4) Cost, in water, for lockage. (5) Cost, in money, of lockage. (6) Cost of locks. Let us see how far these objections militate against a high tide level two-lock canal as compared to a mean-tide-level one-lock canal.

(1) A lock at one end of the Panama Canal would be as much of a handicap on shipping as a lock at each end; if one lock can be made large enough, so can two; there is no limit in length, except that of cost, to which locks may be made, and that cost does not increase arithmetically with the length. The limitation is in width and depth. You can make one lock as wide and as deep as another. So that the one absolutely necessary lock at the Pacific side of the isthmus would be as much of a limitation on the size of ships as two locks would be.

(2) If the waterway is at first properly enlarged immediately next to the locks, a lock-level can be enlarged just as well as a free open level, while in use, after primary completion.

(3) It has been ably shown that slowing up, entering, stopping, locking up or down, and departure, of a large ship (or a lock full of small ones) for a ten-foot average lift, which we are now considering, would be certainly not over thirty, by some authorities only twenty-five, minutes. Supposing the waterway is large enough, the capacity for tonnage per day on the Panama Canal will be limited by lock capacity (or, possibly, by head-way between vessels) and one lock will limit it just as much as two of equal size and capacity and equal lift, when independently placed. The same ship is not passing through two locks at the same time. When she leaves a lock, that tool of the system is ready for another ship, if locks are built in couples and systematically operated. Hence, the handicap, in time, of a tidal lock at the Atlantic as well as the Pacific end would not exceed half an hour per vessel. It will take longer than that time to arrange papers for entry and departure, and that time can be utilized in the lockage if it could be utilized in entering the open end of the canal. Hence, the mean-tide-level canal, with a lock at only one end, would have absolutely no advantage over a high-tide-level canal with a lock at both ends, on the score of time limit on ships or tonnage capacity of canal.

(4) It is by all parties conceded that the catchment basins or lakes deemed necessary in regulation of the Chagres river under any of the plans, when used as storage reservoirs, will afford ample supply to cover the low-water period demands, under any lockage or lock-leakage estimates yet made. Hence, there would be nothing in favor of the one-lock mean-sea canal over the two-lock high-tide-level canal, on the score of greater water consumption.

(5) The cost of lock operation and maintenance has been repeatedly shown to be an insignificant figure per ton on such ships as will use the Panama Canal. Double it for the two-lock high-tide-level canal as against the one-lock mean-sea-level canal, and the alleged expense handicap, of the additional lock, on shipping, will prove to be not worthy of consideration.

(6) The cost of an additional lock for a high-tide-level canal, would, I venture to say, be far less than the alternative cost of carrying the canal excavation, down to a plane admitting of a mean-tide-level waterway. The determination of this point is for those who have the data and are employed for such purpose. I have not forgotten provision for flood escapes from the high-tide plane.

And, so, I would ask those engineers who favor a mean-sea-level canal for the Isthmus, what advantage it would have over a Panama-high-tide-level waterway? I cannot see any that is material, but I can see very material comparative disadvantages for it. It seems to me that I would not like to be a party to putting the National waterway even half way in a hole.

THE ENGINEERING COMMITTEE'S ARGUMENT.

Of course, if it is shown that for the Panama route, a mean-sea-level waterway would have no advantage over a high-tide-level one, then the case of the mean-sea-level is much weakened as against the high-lock-level plan. But the facts must be squarely faced. The Engineering Committee of the Panama Canal Commission, reporting on Feb. 14, 1905, give these reasons for recommending a sea-level canal: (1) "That it could be easily enlarged by widening and deepening at any time in the future, to accommodate an increase in traffic, without any inconvenience to shipping using it." As asserted above, the waterway of lock-level canal can be just as easily enlarged. (2) "A lock canal is in reality a permanent restriction to the volume of traffic and size of ships using it." The committee does not and cannot propose a lock-free waterway for the Panama Canal. Their tidal lock would be as much of a handicap in this connection, as would two tidal-locks commanding a high-water plane. To raise ships to the full lock-lift of say 30 ft. takes only ten minutes longer than to raise them 10 ft., the rate of movement being about 2 to 2½ ft. per minute after the ship is in the lock. So that with a canal having a 30-ft. above mean, or 20-foot above Panama high-tide-plane, the total lockage loss of time would be only 20 minutes more than with the Panama high-tide two-lock plan—an amount of time utterly insignificant in a long voyage.

The committee advanced no argument in favor of its proposed "sea-level" cutting, which I have not shown to be fallacious as between a mean-sea-level one-lock canal and a Panama high-tide-level two-lock canal. Nor have they stated one reason which would apply to make a high-tide-level two-lock canal materially [illegible]rable over a 30-ft.-level two-lock canal.

It may be that some really cogent argume[illegible]d be adduced in favor of a high-tide-level or even [illegible] tide-level waterway, as compared to such high-le[illegible]s as have been proposed by other authorities, bu[illegible] Engineering Committee did not present any such[illegible]ment. The sum and substance of its argument wa[illegible]stan-tially, the following: Because we find by sev[illegible]weeks' steam shovel work on the surface of the gre[illegible] that, in the judgment of the chief engineer, in wh[illegible] con-cur, the cut can be made for $15,000,000 less [illegible]nere-tofore estimated, and can probably be finished [illegible]n or twelve years, and because we find that afte[illegible] there is a good dam site at Gamboa, "this committ[illegible]prove and recommend for adoption by the commis[illegible] plan for a sea-level canal," which "is a waterway w[illegible]o re-striction to navigation." True, nature compel[illegible] to put a lock in it, and "a lock canal is in reality a p[illegible]anent restriction to the volume of traffic and size of [illegible] that use it;" but the sea-level canal will only cost [illegible],000 more than a 60-ft. level one, or $79,742,000 [illegible] more than the 85-ft.-level estimated by our predece[illegible] and will only take four or five years more to constru[illegible], hence, we recommend it.

DANGER IN THE COMMITTEE'S ESTIMATE.—To notice a detail or two: The committee's basis for its earthwork cost estimate seems to have been insufficient for the deduction made from it. To estimate the average for the entire cut, even down where it is narrow and confined and the rock is harder, at about the cost per cubic yard for removing the soft rock and hardened clay, near the present surface, as ascertained by a few weeks' work with one or two steam shovels, is dangerous. To apply that cost as an average for a projected twelve years' work with one hundred steam shovels (a big team to manage on a narrow road) seems to be unsound. Every experienced rock and earthwork contractor will, I believe, recognize that in such figuring there is, to use a contractor's expression, "a lot of room for going broke." There may have been, in the chief engineer's "below fifty cents," as the ascertained cost, by the experimental steam shovel work, a very wide margin on which to predecate his "not to exceed fifty cents" forecast for the whole cut, but the report does not imply it, must less show it.

We might well accept the results of these few weeks' steam-shovel work and the estimate of saving in cost of the canal, based thereon, as sufficient groundwork for the committee's first recommendation, namely: "That this actual reduced cost of excavation justifies a reduction in the elevation of the summit level of the canal by a correspondingly greater volume of excavation." This application of the lesson of the chief engineer's good work and judgment would have involved the $15,000,000 reported as the probable saving, and no more. But, when the probable saving of $15,000,000 is given as a reason in chief for adopting a plan which involves the certain expenditure of $52,462,000 or $79,742,000 more than two other plans reported upon, and with the $15,000,000 saving still to be realized, then it is time to inquire into the basis of the judgment and the judgment which prompts the recommendation. The real question, after all, would be, whether the expenditure of the additional money would be sufficiently justified, from the national point of view, by the greater value of the resulting waterway.

THE GAMBOA DAM PROPOSAL.—As to the committee's appreciation of the Gamboa dam site: It would seem that they found good abutments, and good foundation rock at about 60 ft. below the water plane in the river. On the face of it this would seem to be a fairly good dam site for a masonry dam, but decidedly more information is necessary to show an experienced engineer that it is a fit place to put such a dam as the committee report. Here they propose to raise a dam about 200 ft. above bedrock, "the plan of which will probably require a masonry core with a great mass of earth and rock fill on either side of it, from the waste excavation of the summit cut." The committee further say, that, "the proposed height of this dam from its foundation to its top is far less than is found in a number of masonry dams already built." Yes, it must be answered, but the committee is not proposing to build a masonry dam. It is proposing to build an earth-and-rock-fill dam with a thin masonry core—the core resting on bedrock, and the flanking earth and rock-fill resting on the 50 to 60 ft. in depth of the river alluvial formation on each side of the core. Did the committee intend to say that "the conditions attending the construction of this dam are in no way unprecedented?" The particular dam they describe, on the foundation they speak of, I mean. Are there precedents for it? If so it were well they should be given. There are intelligent engineers who have been through the mill of special dam-building work as well as the grind of hydraulic engineering theory, and great contractors who think they know every rung of the ladder of such experience, who say they would not put up that kind of a dam on that kind of a site, especially in that kind of a c[illegible]ate.

It is the kind of a dam especially requiring an [illegible]nly good foundation, which does not seem to be prese[illegible]d at the Gamboa site. The thin masonry core is to be fo[illegible]ded on bed-rock, and will not settle. The earth and ro[illegible] fill

embankments on each side of it, will rest on the alluvial formation 50 to 60 ft. deep, and presumably will settle. Such settlement tends to separate the earth and rock fill from the masonry core, there being no bond between them. If this occurs on the upstream side, the crack thus formed becomes filled with water and the thin masonry core becomes a water retaining wall, for which it is not fitted. If the crack occurs on the down-stream side the masonry core is deprived of the support of the earth and rock fill on the side it absolutely requires it. This type of dam is bad and dangerous except for specially adapted sites. This does not seem to be the condition at Gamboa, and the alternating of immensely heavy rains and long, dry, intensely hot spells would be peculiarly trying on such a construction. But if the site is suitable for the type, the height of the embankment is excessive for the type, so far as we have precedent. Two hundred feet in depth, of earthwork (counting alluvial foundation and dam embankment), piled up on each side of a thin masonry core! Is it possible that the settlement will be so even that there will not be a separation of the earth and rock fill from, at least, one side of the core, which will remove the support there? I scarcely think it possible. I think such action would be very likely to occur. If it did, the thin core would soon be cracked. If this occurred the dam would stand a good chance of failing.

But, if perchance, the dam escaped defect from the cause of unsuited foundation or excessive height for its type, the committee's manner of construction would endanger it. They say, in expatiating on its cheapness, "the making of the earth embankment on the two sides of the masonry core is simply wasting the material from the summit cut."

Surely, the committee cannot mean just that. "Simply wasting" earth and rock from a cutting cannot be made to build a dam. That material must be carefully placed in thin layers and be moistened, when not moist enough, and be heavily rolled into the mass, layer by layer, else there surely will be enormous and uneven settlement, the masonry core will be left to support itself on one side or the other, or both, and then will be ruptured, and then sooner or later, disaster will follow.

If the chief engineer's estimate, on which the committee made its report, treated that Gamboa dam as merely a waste heap for his Culebra cut, as the committee's report, in effect, says he did, then his estimate is much too low, for it costs money to properly place earth in an earthwork dam, even after it is delivered in cars from a distant cutting. Especially, it costs money to work up an earthwork dam on each side of a masonry core and get good work in both the earth part and the masonry part, and in the joining of the two. You have to ram by hand the earth next to the masonry, you cannot machine-roll closer than about 10 ft.

Moreover, all of the material of each trainload from that cutting probably will not be fit to go into that dam, or, at least, not into the same parts of it, and assorting by carloads will have to be done, which will cost money. And, then, your railway tracks are in the way of scraping the materials down into thin layers and rolling. This work must be done evenly and thoroughly, and cannot be spoken of as "simply wasting the material from the summit cut."

A good dam probably may be built at the Gamboa site, having a masonry or concrete core wall, heavy flanking and outstanding facewalls of dry quarry stone and with wide spaces between the face walls and the core wall properly filled with earth. Materials of the summit cut could be worked into such a dam; and possibly this is the kind of a dam which the chief engineer planned and estimated, but it is not the kind the committee reported.

THE ISTHMIAN CANAL COMMISSION'S POSITION.

Passing now to the Bohio dam proposals: The Isthmian Canal Commission of 1899-1901, first planned and favored a masonry dam on the so-called French site and on the supposition that the maximum depth to bedrock was about 128 ft. Their borings later showed this depth to be "at least 143 ft." Considering this depth too great for an attempt to found a masonry dam or even a masonry core on the bedrock, they then chose another nearby site where the depth was only "128 ft. below sea level." But they changed the type of dam to "an earth embankment with a heavy masonry core carried down to bedrock;" and say, "for reasons of economy that type of dam is preferable to one wholly of masonry upon the new site, and is now adopted." But since then the borings conducted under direction of the commission of 1904-5 have shown that the "greatest depth to rock both at the French site and on that tentatively proposed by the former Isthmian Canal Commission is about 158 ft. below sea level." So that, as it turns out, in the light of this later data, upon the Isthmian Canal Commission's own judgment of its effect, the construction of their type of dam at Bohio is impracticable.

This is the same type proposed by the engineering committee of the 1904-05 commission for the Gamboa site, and, as a type, is open to the same objections. The depth through the alluvial formation to bedrock at Gamboa is about 60 ft. The depth at Bohio is over 158 ft. The total height of the proposed masonry core at Gamboa was to be about 200 ft. The total height of such a core at Bohio would have to be over 258 ft. Although the above-ground part of the proposed Bohio strucutre would be materially less in height than that of the projected Gamboa structure, there is no reason why the Bohio project of the Isthmian Canal Commission is less free from criticism than the Gamboa planning of the engineering committee of its successor commission, except, presumably, on the point of method of building.

FOUNDING THE BOHIO DAM.—As a matter of fact, the first commission's Bohio dam proposal has been sharply and effectively criticised by engineers of the highest standing, in the discussion of Mr. George S. Morison's Bohio dam paper, read before the American Society of Civil Engineers in March, 1902. Mr. Morison, who was himself a member of the commission, referring more especially to securing foundation for and building the core wall, wrote:

It involves novel and untried features. Few engineers, even among those who feel that they could construct it, would be ready to say in advance how the work should be done. The difficulties taken in connection with the climate and other surroundings are enormous.

In the course of this discussion, Mr. Edward P. North, M. Am. Soc. C. E., an engineer of high standing, pointed out that to get the foundation of the core wall proposed by the commission, to the depth of 128 ft. below sea level at the Bohio site where the ground water stood above sea level, the air pressure in th ecaissons would have to be about the equivalent of five atmospheres, except it might be that during the short low-water period of the year the counterbalancing water head might be somewhat pumped down. He did not think it was safe, however, to count on this relief of pressure being sufficiently accomplished to render the plan free from the gravest dangers and probable disasters. Citing examples, he pointed out that men had never yet worked under such an air pressure as would here be made necessary, even in specially cool places, where the temperature of the water was only 40° to 50° F., whereas, at the Bohio site it had been found to be 90°. The consequences, in his opinion, might be expected to rival those said to have attended the French Panama Canal construction work, in the sacrifice of human life.

On the other hand, Mr. William H. Burr, M. Am. Soc. C. E., who had been a member of the Isthmian Canal Commission, controverted Mr. North's opinion as well as a number of points in Mr. Morison's paper. He took the ground that it was probable that the counterbalancing water head could be much pumped down. He would expect it to be reduced to less than 100 ft. of pressure, and he thought the conditions at the commission's selected site justified this expectation, though it was possible that the deepest caissons would have to be carried to bedrock under the full pressure head if the pneumatic process was adhered to. It should be remarked here that Mr. Burr's own description of the alluvial formation to be worked in, which is the most detailed and explicit account thus far made available to the public, seems to establish conditions which would render his hope of pumping down the waterhead, for a number of the caissons, at least, utterly groundless. He writes:

It will thus be seen . . . that there is a probability so strong as to amount to practical certainty that . . . the water of the river is now in free connection with the pervious substrata of sand and gravel.

If this be so, it would seem to be a hopeless task to keep the head of pressure pumped down during the sinking of caissons through those strata. One would suppose that the pumps would have to handle much of the river water besides lowering the reservoired waters in the open gravels of the great valley above the site.

Defending the selection of the Commission's chosen site, as compared to the French site preferred by Mr. Morison, Mr. Burr admitted that the depth to bedrock at the latter was far beyond any depth yet reached by workers under compressed air. But the still later borings made by the Panama Canal Commission's engineers, have shown the depth to bedrock at the Isthmian Canal Commission's section to be very materially deeper still. This would seem to utterly undermine Mr. Burr's defense of the commission's site and plan.

As an alternative and a means to be adopted at such depths as might prove too great for the pneumatic process in getting the core-wall foundation to bedrock, it was proposed to use the method of dredging through open tubes and driving these to the bottom with the water standing in them. But there was a concensus of opinion that the desired result of cutting off water loss, would be very uncertain of accomplishment in this way.

Several other proposals of means for stopping the percolative loss through the deep-lying gravels were suggested, among which was one by Mr. Ed. Duryea, Jr., M. Am. Soc. C. E., which consisted of forcing cement grout into the layer of permeable sand and gravel by means of pipes driven through the overlying materials. This proposal met with favor and Mr. Morison declared that he would certainly try it in connection with his form of dam if he were to build it.

In defending the conclusions of the commission as to necessity for the masonry core wall to bedrock, Mr. Burr summed up:

If it could be assumed that a blanket of impervious clay and sand is uppermost over pervious material, it is in the highest degree probable that the weight of 90 ft. of water above it would in many places produce settlement cracks through which water would readily find its way to the pervious strata below.

If Mr. Burr is right in this deduction, and he probably is, he has furnished the most condemning illustration of the point I have made against the type of dam he and his associates of the Isthmian Canal Commission planned and recommended, and which he was at the moment upholding. The conclusion is inevitable; if under the weight of water in the reservoir there would be such settlement in the bottom as Mr. Burr predicts, then under the weight of the two earthwork parts of the commission's dam, supplemented by that of the water on its upper side, there would be at least equal settlement, and this would destroy the integrity of the dam exactly as I have described it was liable to be destroyed in the Gamboa case.

THE LESSON OF THESE CONFLICTS.

There is a point in all this to which I have been leading. General H. L. Abbott, in his recent book already noticed, several times refers to the thoroughness and care with which the New Company's Technical Committee, its Statutory Commission, its Consulting Engineers, its Chief and Field Engineers had, for a series of years, amassed data for and studied the physical conditions and problems of the Panama Canal. He says: The duty of acquiring the data needful for elaborating a reliable plan "devolved upon the New Company," and it

has been performed in the most thorough manner. Nothing has been taken for granted. An able staff of Engineers was employed, headed by Mr. Louis Choron and assisted by the Comite Technicque, whose names and professional standing are given in Chapter I. Years were devoted to a conscientious study of the problems.

And yet the Isthmian Canal Commission's borings at the French Bohio dam site, showed that the depth to bedrock was so much greater than had been reported by these French Engineers and Commissions that the site was not available for a dam to be put down by the only process which could, within reach of its limits of application, assure reasonably perfect results. And the American Commission, furthermore, flatly and in detail deny an important fact reported by the French authorities, namely, the existence of any continuous and thick bed of clay or other impervious material overlying the pervious gravels, at the site.

In their report the American Commission say of their own examination at this place:

The Commission has devoted much time to the procurement of full and reliable information concerning the foundation upon which this dam must rest.

But after its report was made one of its own members (and not the least of them, professionally), reaffirms the findings of the French authorities, as to the thick bed of impervious material, and condemns the plans of his associate Commissioners. Finally, the last Canal Commission's Engineering Committee prove that on a very important point the Isthmian Canal Commission had not "procured full and reliable information concerning the foundation upon which the dam must rest," notwithstanding the time devoted to it; and, as a consequence, we must conclude, from the reasoning of the Commission itself and that of Mr. Burr, its champion before the Society, that its dam cannot be built.

ONE PROBABLE REASON FOR THE CONFLICTS.—This is a case wherein any engineer of extended personal field experience in exactly such matters is at liberty to draw his own conclusions. It is a case of radical disagreement as to the disclosures made by a lot of borings in an alluvial formation. It happens that the writer hereof, in a long professional career over very much of this world, has frequently had to study just such data, and has hourly and momentarily in person, even in later years, supervised just such examinations by borings. The result has been that he would not take responsibility for any man's report on such an examination except that of a tried assistant whom he had himself thoroughly trained, and he would watch even his work very closely. There is no examination which a civil engineer has to make in which he can more readily go wrong than in that of testing the permeability and quality of deep-lying alluvions. The examination is one to be most carefully and systematically conducted and watched in every detail and in the light of experience and thought.

How many of these thirty or more gentlemen of deservedly high standing in our profession, in expressing their opinions on this Bohio dam foundation material, knew anything definite or of any sufficient extent from their own personal attendance on and study of those borings? I will not say, none; though from my experience of how such commissions work, I think the assertion would be safe. Visiting the site while the borings are in progress and being shown some of the results, does not constitute personal field study of such a problem. Subsequent examination and mechanical analysis of hundreds of material samples taken from the borings by assistants and well borers, and later worry over section diagrams showing the layers traversed, does not make up for the absence of close personal attention to the field work and individual sampling of every hole.

Unfortunately, we have not been told in any of the reports, papers or books, how these boring examinations were made, or under whose momentary supervision, or by what system the percolations and stand of ground waters were studied. The writer has had opportunity to get some information on several of these points from persons who

were on a part of the work, and from others who themselves had been informed by people who knew at first hand; and he has found no reason from such information to form any opinion other than that implied above.

It is a too general fault that our official technical reports are overloaded with trivial details, abstruse calculations, and irrelevant matter, and lack the exact accounts of work, and vitally necessary data, and thorough reasoning on which the judgments of the experts are based.

Another misfortune is that as we older engineers come into demand and higher station and our opinions are sought after, we are generally obliged to leave to others the examinations on which our judgments must rest. The result is that too often our reports are not worth paying for.

What would the report of the greatest mining engineer in the land be worth on a property in Central America, which report was based on samplings, measurings, assays, mill tests and other examinations, made by a lot of cheap assistants, while the expert himself was in New York or London? Nothing! No experienced mine dealer, operator or company promoter would want a report so made, on which to invest a dollar of his own. The mine expert is expected to go himself and personally make the measurements and tests on which his report is to be based. The judgment of the expert is a valuable thing when based on his own personal, first hand knowledge of the facts. Most of all, it is necessary that his experienced judgment should guide in ascertaining the facts; and, so, he makes the examination in every detail, with the assistance of trusted and experienced helpers immediately under his eye. It is unfortunate that Civil Engineers in high position cannot work in this way. It is more unfortunate, sometimes, when circumstances particularly demand it, that they do not.

The Bohio dam foundation controversy is an illustration of our professional misfortune in the above respects. The question of that foundation is one of the most, if not the most, important, of a physical nature, involved in the whole Panama Canal problem. How to spend $150,000,000 of the American people's money depends upon a right appreciation of the conditions in that alluvial formation, if a sea-level waterway is not to be built. Yet, look at the situation. Utter chaos of opinion, and nothing more definite than mere opinion can positively be given by those on whose judgment the world must rely.

I shall not undertake to explain why it is that those who are to take the responsibility of decision and whose judgments are considered to be peculiarly valuable, should have a good knowledge from close personal study of the basic facts, and should have had special experience in such examinations. I am not writing a book or a specification. Sufficient to say that the facts in such cases are at best obscure. Good knowledge of them is best gained by personal intimacy with every little phenomenon attendant on the search for them. Judgment on them comes gradually, intuitively. I have in my own practice had reason to sympathize with the eminent European and American engineers of Panama Canal experiences, who are now in the attitude of those famous knights errants who met on the high road and fought because a certain shield, aloft on a pole, appeared to one of them to be of gold and to the other, of silver, while to the dwarf who was in the bushes it was known to be of brass.

Even after the report of the Consulting Board of 14 of the most prominent engineers in the world, about to assemble next month, has been made, what American can have confidence in the result of building a 90-ft. dam on the Bohio site, should this be determined upon? If a well-selected small committee of the 14 had a year or two in which to make examinations and made them in person, I, for one, would believe (on the responsibility of the committee) that the facts were well and reliably known, and I should respect the judgment advanced upon them by the Committee or the Commission. It might turn out to be wrong, but it would have been well arrived at.

REMOVE THE UNCERTAIN FEATURE.—But it appears to me, in the light of valuable data, that it is neither necessary nor expedient to build a 90-ft. dam at Bohio. It seems to have been unfortunate that the proposal was ever made.

A 90-ft. Bohio dam involves several points of uncertainty which will be difficult to safely and satisfactorily determine. Why encounter them? Why put such a test on a foundation very difficult to get at the facts of? Why resort to the construction of an unheard of dam, on an unsuitable foundation, by a method never before applied under such extreme conditions? Why undertake to make locks with lifts of 45 ft., when 35 ft. had theretofore been regarded by all specialist engineers as about the safe limit? The plan of the one great Bohio reservoir as flood regulator, water supplier and navigable lake has never been shown by argument or comparison of plans and estimates made under comparable conditions, to be better or materially cheaper than the two-lake proposition of the New Panama Canal Company. What is the objection on the part of any one who would favor the 90-ft. Bohio lake project, to some plan very nearly like the 66-ft. Bohio lake, coupled with the Alhajuela reservoir, proposed by the French Commissions?

COMBINE THE KNOWN SOUND FEATURES.—So far as the writer has been able to ascertain, no fault has been found with the Alhajuela dam site or reservoir site; (1) except that it is a little remote for cheap construction of a masonry dam; (2) is too high up to act as a sole regulator for the Chagres; (3) cannot be worked in with a project for spill-way tunnels through mountain ridges; and (4) the dam cannot be used as a spoil bank for the Culebra cut work. But, the first objection is a matter for comparative estimates, which have not been made by one authority on one basis; the second objection does not stand if the reservoir is to be coupled with a 66-ft. Bohio lake; the third will not hold if we don't have to have the spill-way tunnels; and the fourth need not be seriously considered.

Moreover, if the late Chief Engineer and the Engineering Committee are right in their facts and judgments as to the Gamboa dam site, then, substitute the Gamboa reservoir for the Alhajuela, in the project with a 60-ft. Bohio lake as a summit level. As the Engineering Committee has demonstrated that the Gamboa reservoir with a 140 to 150 ft. dam, above ground, and proper spill-way provision, would, by itself, serve to regulate upper Chagres flood flow; and as the New Panama Canal Company's engineers have well demonstrated that the 66-ft. Bohio lake, coupled with the Alhajuela reservoir, would amply serve as a middle as well as upper Chagres regulator; it would appear that a 60-ft. Bohio lake, combined with about a 120 to 130 ft. (above ground) Gamboa lake, would amply serve the full purpose. This combination would meet the recommendations (1) of the New Company's Commissions and the Isthmian Canal Commission in favor of a lock-level work and in condemnation of a sea-level idea; (2) it would meet the recommendation of the New Company's Commission, on the point of a 66-ft. Bohio lake summit level combined with an upper Chagres reservoir; and (3) it would meet the recommendation of the Engineering Committee of the last Commission, on the point of elevation of summit level and utilization of Gamboa site as the upper Chagres regulator. There certainly is authority enough to commend this combination as against the Isthmian Canal Commission's divided opinion on the 90-ft. Bohio lake project, and as against the sea-level proposal of the Engineering Committee.

THE PUBLIC AND THE ENGINEERING PROFESSION.

It has, unfortunately, happened that the action of both of our American official authorities has been calculated more to upset the faith of the profession and the public in any Panama Canal project than to establish a firmer faith. This is an unavoidable conclusion. Neither of these groups of engineers, so far as we can judge by the published record, treated the majority opinions of those who preceded them with prudent consideration. The plannings already formulated seem in both cases to have been virtually consigned to the waste-basket. Each Board appears to have ruthlessly cast aside the real conclusions of their predecessor experts, who certainly were of deservedly high professional standing, and to have immediately set about finding a new and original way of doing the Panama Canal job. Now, it may have been expedient that further studies, plans and estimates be made, in each case, but the presentation of results should have incorporated some sufficient reasons for the abandonment of that which had been arrived at before. One set of professional men cannot, practically, snap their fingers in the faces of another set of equal station and authority, without the public losing some part of its respect for the opinions of both.

TOO MUCH MERE ENGINEERING OPINION.—The Engineering Committee may well plead that they were only a committee reporting recommendations to their Commission, and that, as it turned out, the Commission did not have opportunity to present their case fully in an extended report for publication. That seems to have been their misfortune. But the Isthmian Canal Commission of 1899-1901, as a Board of Engineers, can make no such defense. The only hint of a reason given in their report for not adhering more nearly to the project which had been formulated under the guidance of 15 to 20 engineers of foremost standing in connection with such matters, in France, Germany, Holland, Great Britain, the United States and several other countries, was that the question of time for completion of a work would be of less importance to the United States than it had been to the French Company; that the United States could finance such a venture to better advantage; that a government, seeking "the permanent development of its possessions, and content to receive its returns in an indirect way and at a future time, must build a canal of dimensions which will permit the passage of the largest ships now afloat or likely to be constructed." Questions which might circumscribe the action of a commercial company in choosing a plan, with a great government "would not be decisive against a plan otherwise preferable," the Commission wrote.

Undoubtedly, so, it may be answered, but, why, where or how is the plan submitted by the nine Isthmian Canal Commissioners "preferable" to that formulated and preferred by the fourteen engineers of the Technical Committee, and the five members of the Statutory Commission, who preceded them in the study. The advisability of increasing the capacity or size of the waterway was no more a reason for throwing over a plan in [illegible] case than was the probable saving of $15,000,000, a [illegible] for expending an additional $52,000,000 or $70,000,[illegible] the case of the Engineering Committee's recommendation. Unquestionably, a canal could be built by the [illegible]-level project of the Technical Committee to as great [illegible]acity as by the 90-ft. level project of the Isthmian Canal Commission. It might have been that the proj[illegible] the Isthmian Canal Commission was preferable to [illegible] of the Technical Committee, but the proposition was [illegible] the assets of the New Panama Canal Company, a[illegible] of those assets were the results of the expenditure [illegible] much money on engineering investigation and advi[illegible]. The Isthmian Canal Commission not only ignored the [illegible]sions and concrete results of the expert work of [illegible] high engineering authorities who preceded them, [illegible] deliberately threw over an asset which it recommended the government to purchase, without giving any re[illegible] that they had formulated a plan which was "preferable."

THE ORIGINAL AMERICAN PLAN—CANALIZATION OF THE VALLEY AND SWAMP AREAS.

So far as the official reports show, the advantage does not seem to have been considered of, at one and the same time, (1) limiting to a minimum the depth and volume of under-water work, (2) eliminating intercepting levees and diversion canals, and (3) abolishing the two great terminal swamp areas. Starting in with the idea of an open waterway from ocean to ocean, and apparently impressed with a sense of the advantages of such disposition if it were possible to realize it as a whole, the engineers all seem to have conceded that as much of the length of the route as favorable for application of that idea should be given over to it. To cut and dredge as far as Bohio from the Atlantic and as far as Miraflores from the Pacific, for the most part through very low if not actually swampy ground, was favorable for a sea-level construction. M. de Lesseps rushed construction on this, and thereby embarrassed later engineering decisions. The control of Chagres floods, the Bohio dam possibilities and the Culebra cut problems seem to have held the attention of later authorities. All of them appear to have attached to consideration of a dam below Bohio, for instance, an overwhelming sense of the difficulties of successfully building and maintaining a high dam on a bad foundation. So that every proposal to put a dam at a lower point, to make any level of lake navigation below Bohio, has been ignored or rejected. It would seem from the published record that this has been done on but slight consideration and investigation. Perhaps the facts warrant such course, but those made available and the remarks based upon them in the reports, seem insufficient to justify it.

TERMINAL LAKES IDEA NEVER STUDIED.—It may be that conditions are not such as to admit of these terminal lakes. But, as yet, we have not been told so as matter of professional opinion, much less has it been demonstrated that they are impracticable. Furthermore, it may be that the advantages to be attained by incorporating such lakes into the Isthmian waterway, as compared to some disadvantages or risks attendant upon such adoption, are not great enough to warrant their construction. This point has not been reported upon.

We may understand that the engineers of the Original Panama Canal Company, at first carried away by the sea-level canal idea, were afterwards anxious to retain all of the original plan and accomplished work possible, and, further, that the engineers of the New Panama Canal Company, heir to the results of the errors of the original company, also strove to make use of as much of the old work as was possible, it being an asset to be conserved. Hence there was no French authority likely to take up the terminal lakes idea with interest.

Next, we may understand that our Isthmian Canal Commission, including the independent member, Mr. Morison, having in mind "a location for the dam by which the summit level must be maintained" (using his words) and having under consideration an 82 to 90-ft. summit elevation, decided that "no location suitable for a dam exists on the Chagres river below Bohio."

And, finally, we can understand that the last Engineering Committee, apparently, thinking of a site to be used in lieu of that at Bohio, and regarding a bedrock foundation as essential for that purpose, came to the conclusion that "It is not feasible to construct a dam across the Chagres river at any point lower down in its course that at Bohio," and this, even though they made no borings below Gatun.

THOROUGH CONSIDERATION ADVISABLE.—Accepting condemnation of the idea of maintaining anything like a 90-ft. or a 66-ft. or even a 45-ft. water-level in a Chagres valley lake by a dam at any point below Bohio, we have not been shown that a 25 or a 30-ft. level may not be maintained, nor has it been authoritatively shown or said that such a terminal lake cannot be made by a dam at La Boca, on the Pacific side.

A foundation has to be extremely bad whereon, with an ample fund at disposal, a dam to retain water 25 to 30 ft. in depth cannot be safely placed and maintained. There are many dams, embankments and levees on very bad foundations in this world which have long held water to such depths of pressure, and are expected to hold it forever. The depth to bed-rock under them is unknown, and no one ever bothered about the point. There are

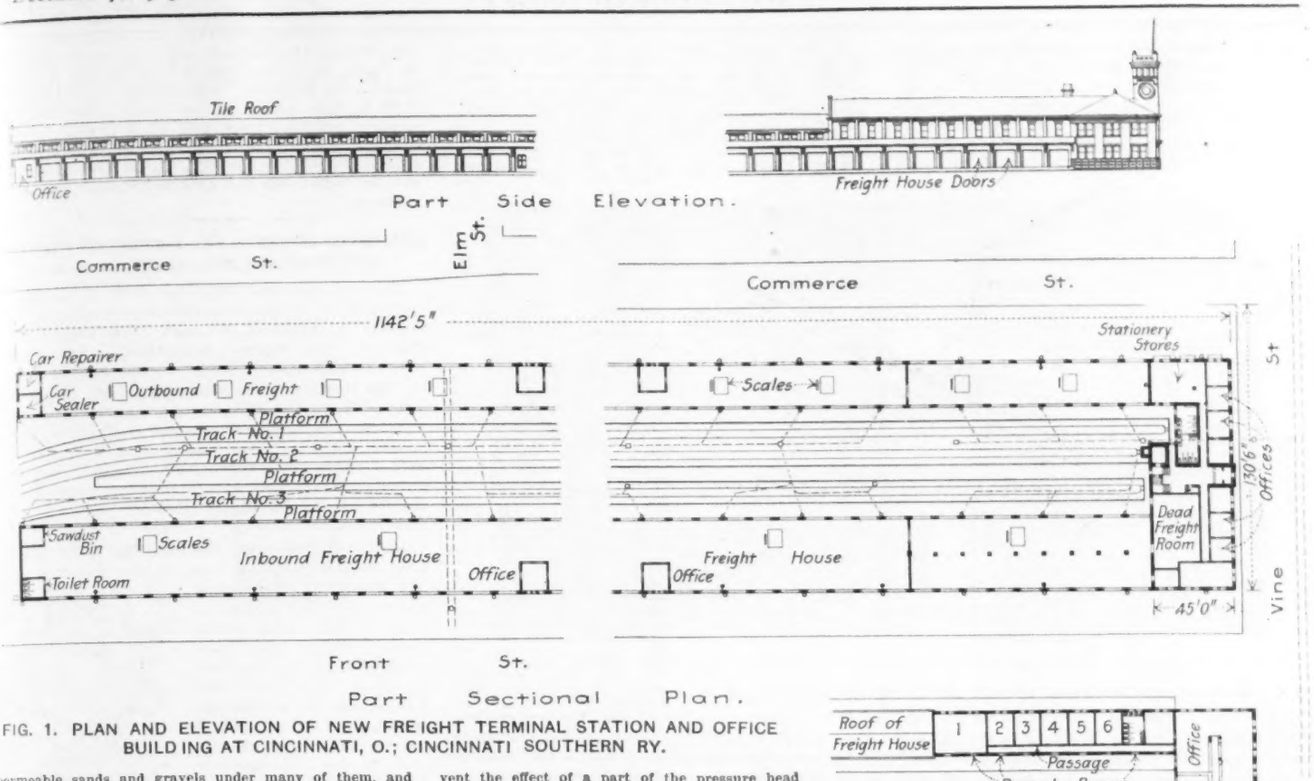
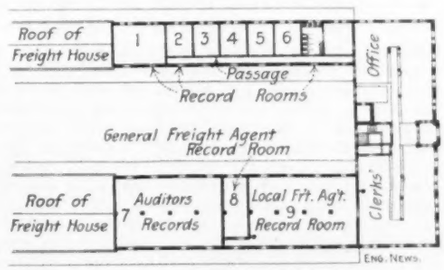


FIG. 1. PLAN AND ELEVATION OF NEW FREIGHT TERMINAL STATION AND OFFICE BUILDING AT CINCINNATI, O.; CINCINNATI SOUTHERN RY.



Part Plan of Upper Floor.

permeable sands and gravels under many of them, and yet they are practically water-tight. There are soft alluvions under many, and yet they have for very many years proven to be safe. Nearly all great sea coast dykes, tide land embankments, swamp land reclamation and river levees, afford cases in point and there are many reservoir dams standing for very many years under similar conditions. If for such works we had to go to bed-rock for foundation or to cut off threatened percolation, they could never be built. And, therefore, I repeat, the conditions would have to be remarkably bad which would justify the assertion that dams or embankments to hold water under 20 to 30-ft. heads, cannot be built and maintained across the rivers in the lower part of the Chagres and Rio Grande on the Panama Canal route. It is to be supposed, of course, that firm foundation is to be had adjacent to the dam embankments, or elsewhere, for the attendant wasteways, sluices and locks.

ADVANTAGES OF TERMINAL LAKES.—Lakes made to depths of 20 to 25 ft. above mean-sea, by works at La Boca on the Pacific and Boca Mindi on the Atlantic (and at a corresponding point across the Chagres, of course), in consonance with the basic ideas of our best canal engineers of the period when our canals were built, would have marked advantages as features of the Panama Isthmian waterway.

They would be fresh water lakes, parts of which could be easily converted into convenient harbors almost immediately at each end of the route.

They would immensely simplify the handling of flood volumes in the lower valleys traversed by the waterway.

They would add much to lake navigation and diminish restricted canal navigation, thereby diminishing time of transit, for the channels could be cheaply dredged to 35 or more feet in depth through them.

They would drown out the two largest and worst swamp areas and pestilence breeders.

The lower Chagres lake would counterbalance and prevent the effect of a part of the pressure head tending to create percolation under a Bohio dam.

They probably would materially reduce the cost of construction as well as maintenance of the navigable waterway. But this is a point for estimate on exact data. It happens that much work has been done towards the excavation of a sea-level canal, which these lakes would submerge, and to some extent this may be a point against them. There probably are other objections to this terminal lakes idea, but we ought to see them lined up fairly and fully with the advantages, before the original and distinctly American idea of the Panama waterway is either thrown over or radically departed from. A large sum of money is to be expended within these terminal divisions of the canal. It would be well to have some plain and convincing evidence that it is to be expended by the best plan.

All of the foregoing is written for the purpose of showing that the Panama Canal problem has, so far as any printed report bears evidence, never been argued out, and a solution demonstrated so as to satisfy any intelligent, attentive and practical person that the best or even a good solution has been arrived at. If we are to take the opinion of one set of engineers only to have it ignored and ridden over by the next, in delivering themselves of another, the agony will never end. Much more is necessary than the mere recommendation of any one plan, no matter who it comes from. Those who have been named by the President to advise as to a plan, have it in their power to lay a proper foundation on which the Isthmian Canal Commission, the President or Congress may wisely determine as to the best plan to build upon.

NEW FREIGHT STATION AT CINCINNATI, O.; CINCINNATI SOUTHERN RY.

Among the improvements to the railway freight terminal facilities at Cincinnati, O., is the construction of a large concrete freight house for the Cincinnati Southern Ry. At one end of this is a two-story fireproof building for the freight offices and also for the handling and storage of records for the various departments, which are thus disposed of more economically than by the renting of large vaults in the building where the company has its general offices. The building has a frontage of 130 ft. 6 ins. on Vine St., and extends west, lying between Commerce and Front St., and having a total length of 1,142 ft. 5 ins. It was designed by Mr. George B. Nicholson, M. Am. Soc. C. E., Chief Engineer; assisted by Mr. H. E. Warrington, Assoc. M. Am. Soc. C. E., and Mr. A. Ritter, Assistant Engineers. The contractor was the Collier Bridge Co. The building and its accessories will cost about $250,000, and will probably be ready for service by April, 1906.

The general design and construction are shown by Figs. 1 and 2. The headhouse is a two-story building, with clock tower; it is of reinforced concrete, faced with red pressed brick, and having a red tile roof. This fronts on Vine St. On the

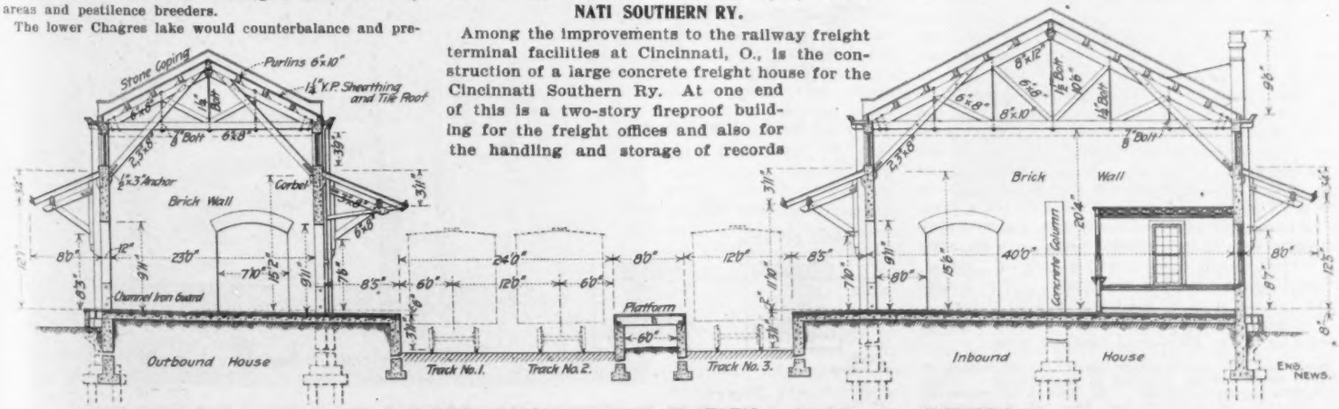


FIG. 2. CROSS-SECTION OF FREIGHT STATION; CINCINNATI SOUTHERN RY.

first and second floors of the main building, 45 ft. deep, are the freight offices, while on the second floor of the two wings are the offices for the records of the following departments: 1, General manager; 2, purchasing agent; 3, vice-president; 4, passenger agent; 5, chief engineer; 6, car accountant; 7, auditor; 8, general freight agent; 9,

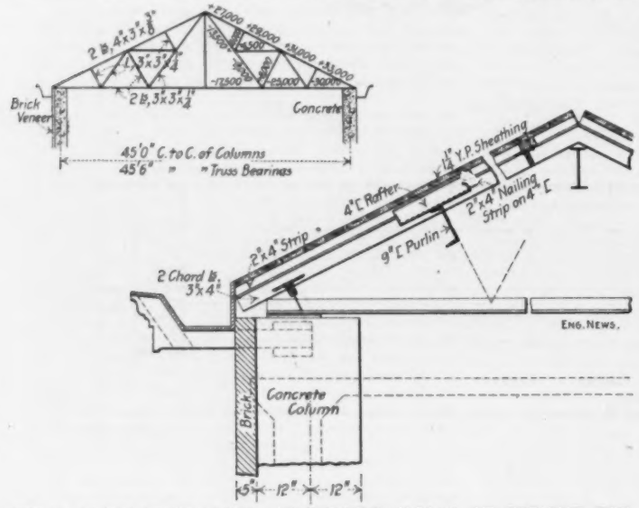


FIG. 3. STEEL ROOF TRUSS OF OFFICE BUILDING AT FREIGHT TERMINAL; CINCINNATI SOUTHERN RY.

local freight agent. The inbound freight house is 42 ft. wide, with a 2-ft. platform and 20-ft. roadway on the street side and an 8½-ft. platform next to the inbound freight track. An 8-ft. trucking platform separates this from the two outbound freight tracks. The outbound freight house is 23 ft. wide, with an 8½-ft. platform on the track side and a 2-ft platform and roadway about 40 ft. wide on the street side. The tracks enter the station on a curve of about 350 ft. radius. The foundations, walls and floors are of concrete, the floors having a wood covering, and the walls of the two-story office building being faced with red brick. The roofs are of tile, supported by wooden trusses in the freight houses and by steel trusses in the office building. All the buildings are lighted by electricity.

The freight houses are divided midway of their length by brick firewalls, extending above the roof and fitted with automatic sliding fire doors on each side. Similar walls separate them from the lower story of the office building. The sides are almost entirely open, consisting of doorways 11 ft. 6 ins. wide and 10 ft. high, separated by piers 3 ft. 6 ins. wide and 20 ins. thick. These doorways are fitted with rolling shutter doors of galvanized corrugated steel. There are 16 scales in the outbound and 8 scales in the inbound house, and each house has two offices for the checkers, etc.; there are also small offices for the car repairer and car sealer. The walls are of concrete, reinforced with steel bars of the Kahn pattern and are 12 ins. thick, with an 8-in. pilaster in the pier between each pair of doorways. The walls support timber trusses of the construction shown in Fig. 2; these trusses are spaced 15 ft. c. to c., and carry purlins with plank sheathing for the tile roof. Timber brackets support the roofs over the platforms, and these also are covered with tile. In the concrete floors are laid wooden stringers, to which are spiked a ⅞-in. yellow-pine flooring and a top surface of 1⅛-in. maple, both laid diagonally. The floor of the outbound freight house has a fall of 6 ins. to the edge of the platform, while that of the inbound house has a fall of 8 ins. to the edge of the street platform and its track platform has a fall of 2 ins. in 8½ ft. The trucking platform has 12-in. concrete walls with 2½ × 12 in. joints, 16 ins. c. to c., and a floor of 2 × 6-in. dressed oak plank laid diagonally. The height from rail to top of all platforms is 3 ft. 11 ins.

The specifications include the following requirements as to the reinforced concrete work:

Steel used in reinforcement must be so shaped or formed as to prevent slipping in the concrete. Plain bars will not be permitted. No system of reinforced concrete will be accepted that does not make proper and sufficient provision for the shear as well as bending in beams, girders, etc.

All slabs, beams, girders, etc., of the floors shall be figured for both the dead and live load. The live load to be taken as follows: Office building, first floor (except stationery room) and second floor, 125 lbs. per sq. ft.; stationery store room, 300 lbs.; attic floor, 100 lbs. Record rooms over freight houses: second floor, 300 lbs.; attic floor, 100 lbs. Columns and footings shall be figured for the full floor load above specified, and also for three additional floors with 100 lbs. per sq. ft. on each.

Bearing allowed on soil, 3,000 lbs. per sq. ft. Concrete: Compression, 500 lbs. per sq. in.; shear, 50 lbs.; adhesion to steel, 50, lbs. Concrete will not be figured for tensile stress. The steel is taken at 16,000 lbs. per sq. in. for tension. Columns will be figured for compression on the basis of the concrete area only, the steel reinforcement being figured only for the eccentric stresses on columns due to girders, etc.

Steel shall be made by the open-hearth process, and shall have an ultimate strength of 60,000 to 70,000 lbs. per sq. in. Elastic limit, not less than 35,000 lbs. It shall contain not more than 0.05% of phosphorus or sulphur. In all other respects it shall comply with the Steel Manufacturers' standard specifications for such grade of steel. Before being embedded in the concrete, the steel must be thoroughly cleansed of all oil, dirt, grease, or other matter that may be detrimental to the positive adhesion of the concrete to the steel. A slight film of red rust will be permitted, but any bar on which rust scales have begun to form will be rejected, unless such scales are thoroughly removed with wire brushes. No bars for this work will be painted or oiled.

For the facing of the interior and exterior walls of the inbound and outbound houses the stone in the concrete was spaded back from the face, and after the removal of the forms the surface is to be tooled or floated. The walls, floors, stairs, etc., of the office building are all of monolithic construction, with interior plaster finish for the walls, and slate slabs for the stairways. The fireproof partitions are of Mackolite blocks. The roof trusses are of the design shown in Fig. 3, with 9-in. channel purlins riveted to the diagonals and carrying rafters of 4-in. channels with nailing strips for the 1¼-in. plank sheathing.

For the present this terminal will be reached from the main line in the west end of the city by means of existing surface tracks of other railways. Eventually, however, a viaduct is to be built leading south from the elevated approach to the Cincinnati Southern Ry. bridge over the Ohio River (at a point near 8th St.) and reaching the street grade at the intersection of Front and 3d Sts., whence the company will have its own tracks in Front St. to the west end of the new freight terminal at Plum St. The viaduct will be 8,200 ft. long, with steel deck spans of 55 to 85 ft. span on concrete piers, and three through truss spans of 105 ft., 118 ft. and 160 ft. It will have grades of 0.95% and 1.68%. Steps have been taken for obtaining the ground required for the viaduct.

REINFORCED CONCRETE PILE FOUNDATION FOR LATTEMANN BUILDING, BROOKLYN, N. Y.

The accompanying drawings show a building foundation of reinforced concrete piles constructed recently in Brooklyn Borough, New York City. This is the largest foundation work using reinforced concrete piles to be carried out in this country. The foundation area is about 100 × 129 ft., and 480 piles were used, driven in clusters of generally four piles for column and pilaster footings. Each cluster of piles is capped with a reinforced concrete slab. The following is a description of the work from information furnished by the Corrugated Concrete Pile Co., of New York City:

DESCRIPTION OF PILES.—The piles used were of the corrugated form recently invented by Mr. Frank B. Gilbreth and shown in cross-section by Fig. 1. In longitudinal section these piles have a uniform taper from butt to point and have flat points; for the work illustrated the piles are 16 ins. in diameter at the top and 11 ins. in diameter at the point. Each pile is cored in the center, the core being 4 ins. in diameter at the top and 2 ins. at the bottom end. Through this core the water jet is inserted for sinking the pile into place.

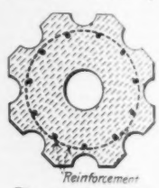


Fig. 1. Cross-Section of Corrugated Reinforced Concrete Pile.

On each pane of the octagon sides of the pile is a corrugation whose section is a segment of a circle, usually from 2½ ins. to 3 ins. in diameter. The principal object of these corrugations is to give passages for the escape to the surface of the water forced through the center core hole in driving the pile. They are also for the purpose of increasing the perimeter of the pile and thereby gaining greater surface for skin friction. In addition to this corrugation increasing the perimeter of the pile, it is claimed that this reduction in area, together with that made by the core in the center,

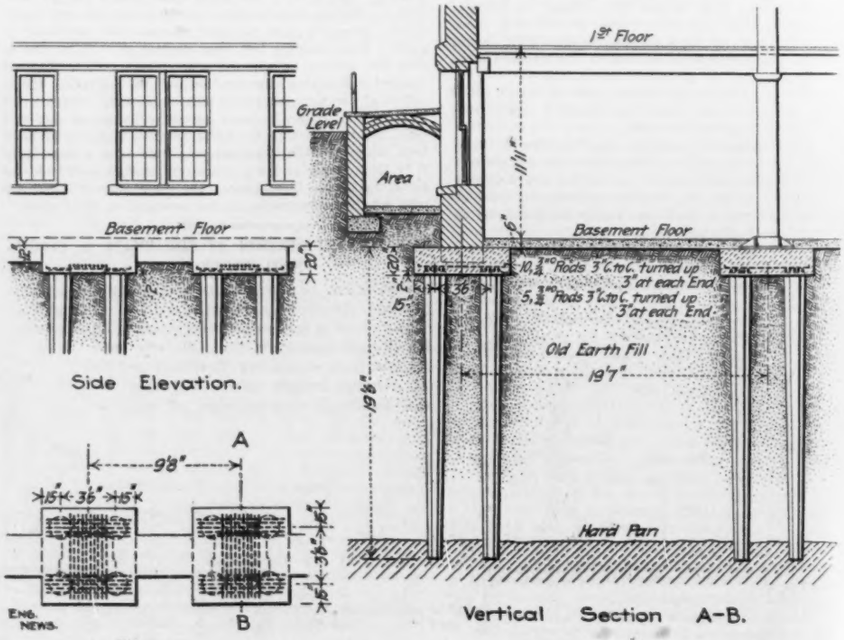


FIG. 2. DETAILS OF REINFORCED CONCRETE PILE FOUNDATIONS FOR LATTEMANN BUILDING, BROOKLYN, N. Y.

ermits a greater surface for skin friction than an be had for any other form of pile with the ame cubical contents.

The piles are reinforced longitudinally and ransversely. On this particular job the reinforcement was formed with Clinton Electrically Welded Fabric, the meshes being 3 ins. × 12 ins.; the longer dimension being lengthwise with the pile and of No. 3 wire; the horizontal or transverse reinforcement being of No. 10 wire. The meshes being electrically welded together, the reinforcement was got out from a wide sheet taking the form of a cone. No part of the reinforcement was closer than 1 in. from the outside of the concrete. In general only sufficient sectional area of material is put in the reinforcement to take the tensile stresses caused by the bending action when handling the pile preparatory to driving; more reinforcement than this only being necessary when the piles are used for wharves, piers or other marine structures, where a considerable length of pile is not supported sidewise, or where they are subjected to bending stresses, which would obtain if the piles were used in connection with ferry slips or landings.

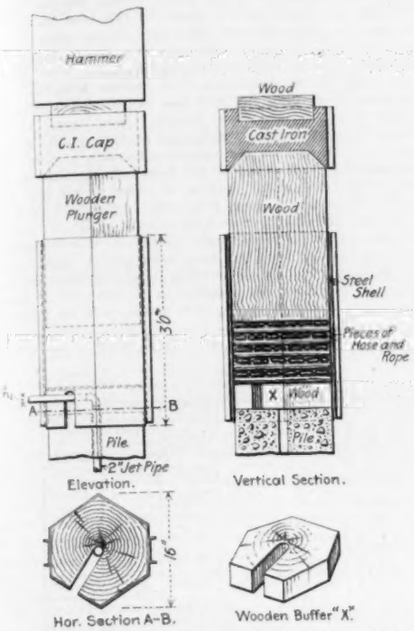


Fig. 3. Sketches Showing Cap Used in Driving Corrugated Concrete Piles.

MOLDING OF PILES.—The forms for molding the piles are made with 2-in. stuff gotten out to the required dimensions, the corrugations being formed by nailing pieces on the inside whose section is the segment of a circle. The sides of the octagon are fastened to the ends through which the core projects some 6 or 8 ins. At times while the molding of the pile is in progress, the central core is given a partial turn to prevent the setting of the cement holding it fast and thereby preventing the final removal.

The stripping of the forms from the piles is usually done from 24 to 48 hours after molding, and from this time on great care is taken that there is a sufficient amount of moisture in the pile to permit of the proper action for setting of the cement. This is usually accomplished by covering the piles over with burlaps and saturating with water from a hose; the operation of driving the pile not being attempted until the concrete is at least ten days old.

DESCRIPTION OF FOUNDATION.—As shown by the drawings, the outside walls of the building are composed of piers with curtain walls between them. The loads are centered on the piers, each of which is carried on a group of four, five or six piles, according to the load, spaced 3 ft. 6 ins. c. to c. These piles sustain a load of about 25 tons each. The drawings show clearly the reinforcement of the pier footings and the arrangement of the curtain wall footings. They also show the arrangement of the interior column footings and of the piles supporting them.

The nature of the ground on which the building is founded is dry fill of from 15 to 20 ft. in depth overlying hard pan. The regulations of the building department demanded that the foundations be carried down to hard pan. The average depth of the hardpan is at or near the level of high tide, so that wooden piles which require to be submerged were not permissible. Figures were taken for masonry piers carried down to hard pan, but this method was abandoned in favor of concrete piles on account of the difference in cost. To ascertain the depth at which hard pan would be reached and to find the nature of the dry fill preparatory to driving, some 30 holes were bored at different parts of the site. This was accomplished by using a 3-in. pipe with a 1-in. pipe placed inside of it in the form of a jet. In this way the nature of the soil at the different depths, and the points at which hardpan was encountered, was ascertained.

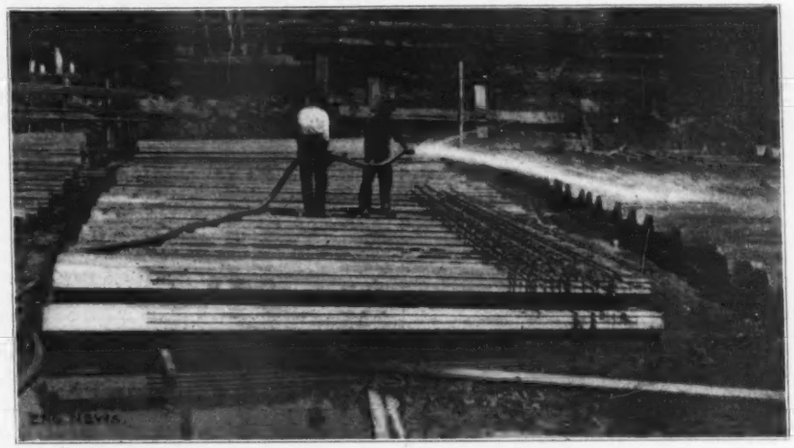


FIG. 5. VIEW SHOWING CURING OF CORRUGATED REINFORCED CONCRETE PILES.

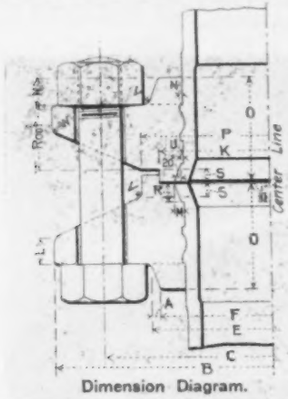
DRIVING THE PILES.—The operation of driving corrugated concrete piles is somewhat similar to that for driving ordinary wooden piles by water jet, but a much heavier hammer with less drop is used. The jetting is accomplished by inserting a 2-in. pipe within the pile. This pipe is tapered at the bottom end to 1-in. diameter, forming a nozzle, and the water pressure used is about 120 lbs. per sq. in. As a rule, this pressure is obtained by the use of a steam pump which may be connected with the boiler which operates the pile driver, or with a separate steam supply. At the upper end of this 2-in. pipe an elbow is placed and a short length of pipe is connected to this and to the hose from the water supply.

Fig. 6. View of Driver and Jet Arrangement for Driving Corrugated Reinforced Concrete Piles.

As it is not practicable to drop the hammer directly on the head of the concrete piles, the driving is accomplished by the use of a special cap. Fig. 3. This cap is about 3 ft. in height and the bottom end fits over the head of the pile.

FIG. 4. VIEW SHOWING FORMS FOR MOLDING CORRUGATED REINFORCED CONCRETE PILES.

In one side of this cap is a slot from the outside to the center, which permits the 2-in. pipe, which supplies the water jet for driving the pile, to project. The outside of this cap is formed with a steel shell, the inside has a compartment filled with rubber packing and the top has a wooden block which receives a blow from the hammer. In this way the head of the pile is cushioned, which prevents the blow of the hammer from bruising or breaking the concrete.



Dimension Diagram.

During the operation of driving, the water from the jet comes up on the outside of the pile and carries with it the material which it displaces in driving. This, with the assistance of the hammer, allows the pile to be driven in place, and, contrary to what might be supposed, after the operation of driving when the water has saturated into the ground or been drained away, this operation puddles the earth around the pile, so that after a few hours' time the skin friction is much more than it would be with the pile driven into more compact soil without the use of a jet.

In connection with the drawings of the Lattemann Building shown in Figs. 1 and 2, we show three views taken on another job and illustrating the methods of molding, curing and driving reinforced concrete piles of the form being described. Fig. 4 shows the character and arrangement of the forms for molding. Fig. 5 is a view of a number of molded piles laid out in rows for sprinkling and curing. Fig. 6 shows the arrangement of pile driver and water jet during driving.

A UNIVERSAL STANDARD FLANGE LIST FOR MARINE WORK.*

By Luther D. Lovekin.†

The subject of flanges for all purposes where pipes have to be connected by such means is one of the most important details in connection with modern engineering and one which, without doubt, has received the least attention among designing engineers. While prominent engineers have frequently taken the subject up, they have seldom given the matter serious thought, but instead, as a rule, when a matter of this kind has been brought to their attention, they have simply asked where the trouble was, and in the case of a flange springing between bolts, or the bolts themselves being weak, they have said to the draftsman "Put another ⅛-in. on the thickness," or "Put the next size larger bolts in, and I think you will avoid the trouble. How many engineers have ever taken the trouble to analyze the forces set up in flanges of the various sizes with a view to determining what would be the most suitable shape of a flange, the amount of surface a gasket should have in order to produce a good joint and, furthermore, to see that whatever flange was used would, as far as practicable, be of the least weight consistent with strength? It seems to me there has been more talk on such items as multiples of four bolts being absolutely essential to any standard, and the thoughts as to how this or that suggestion would affect the valve maker, with his numerous patterns on hand, than as to how the whole matter was going to affect the engineer at sea or on shore, and the owners of our large vessels and industrial works.

There is no doubt in my mind but that the industrial establishments throughout our whole country are suffering from leaky joints in steam pipes, and spending thousands of dollars on various kinds of gaskets or other jointing material in order to remedy defects which are not always due to the kind of material used for such gaskets, but are chiefly due to a lack of strength in the flanges themselves. The writer firmly believes in standards, and appreciates the fact that others before him have made standards, but appreciate the additional fact that no good standard flange list appears to be in general use at the present time for the higher pressures; in fact, some of our best concerns in New York City to-day are ordering forged-steel flanges of the same dimensions as the standard cast-iron ones in use at the present time for large power plants. Surely they are not doing this un-

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†Chief Engineer of the New York Shipbuilding Co., Camden, N. J.

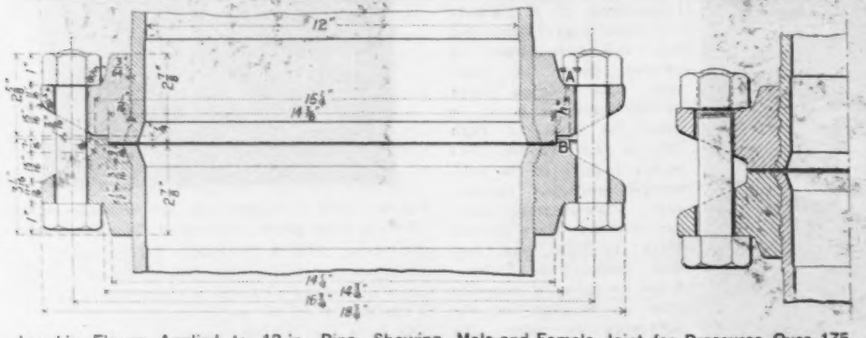
DIMENSIONS OF PIPE FLANGES FOR UNIVERSAL STANDARD

100 LBS. TO 300 LBS. PRESSURE

FOR PRESSURES BELOW 175 LBS. THE MALE AND FEMALE JOINT MAY BE OMITTED AS SHOWN BELOW

L. D. Lovekin April 27, '15

NOMINAL DIAMETER OF PIPE	T. THICKNESS OF PIPE	B. OUTSIDE DIA. OF FLANGE	INSIDE DIA. OF FLANGE	K. OUTSIDE DIAM. OF GASKET	P. DIAMETER OF SPIGOT	S.	O. DEPTH OF FLANGE FROM JOINT FACE	H. DEPTH OF HUB	F. MINIMUM DIAM. OF HUB	E. MAXIMUM DIA. OF HUB	L.	C. DIAM. OF CIRCLE	NUMBER OF BOLTS	R.	M.	U.	N.	V.	W.	BURSTING PRESSURE LBS. PER SQ. IN. ALLOWING A 50000 LBS. TENSILE STRENGTH	LOAD ON EACH BOLT DUE TO BURSTING PRESSURE IN PIPE	STRESS AT BOTTOM OF THREAD IN BOLTS DUE TO BURSTING PRESSURE WHEN AREA OF PIPE TO UNDER DIA. OF FLANGE	STRESS ON BOLTS TO GIVE 100000 LBS PER SQ. IN. PRESSURE ON GASKET	STRESS IN FLANGE DUE TO MOMENT OF LOAD ON BOLT AT BURSTING PRESSURE IN PIPE
2	.176	6⅛	2⅞	3⅛	3⅝	¼	⅝	1½	8	⅜	3½	3⅛	4	⅝	4¾	4	⅝	⅝	¾	9375	9850	48600	4570	18500
2½	.191	6⅞	2⅜	3⅝	4⅛	¼	⅝	1⅝	8.3	⅞	3⅝	3⅞	4	⅝	5¼	6	⅝	¼	⅝	7500	7430	36800	3620	18300
3	.209	7¼	3⅜	4¼	4⅜	¼	⅝	1⅝	8	⅞	4⅛	4⅝	4	⅝	5⅞	6	⅝	¼	⅝	6770	9750	48300	4595	18500
3½	.218	7⅝	4	4⅝	5⅜	¼	⅝	1⅝	7.7	1	4⅜	5	8	⅝	6½	8	⅝	¼	⅝	6250	8920	44200	4100	18200
4	.231	8½	4⅜	5⅜	6	⅜	⅝	1⅝	7.5	1	5½	5⅞	8	⅝	7⅛	8	⅝	¼	⅝	5850	10700	53000	5000	20500
4½	.244	9½	5⅜	6	.137	⅜	⅝	1⅝	7.25	1	6⅛	6¾	8	¾	7⅞	8	¾	⅜	¼	5550	12600	41800	4370	20700
5	.258	10⅛	5⅜	6½	1.04	⅜	⅝	1⅝	7	1	6⅞	7½	8	¾	8½	8	¾	⅜	¼	5310	15100	59000	4560	24700
5½	.271	10⅞	6⅜	7	1.04	⅜	⅝	1⅝	7.3	1	7⅝	8¼	8	¾	9⅛	8	¾	⅜	¼	4830	16400	54200	4710	24400
6	.284	11½	6⅞	7⅝	1.2	⅜	⅝	2	7.1	1	8⅜	8¾	10	¾	9⅝	10	¾	⅜	¼	4687	15000	49500	4550	21400
6½	.297	11⅞	7⅜	8⅛	1.	⅜	⅝	2⅛	6.9	1	8⅞	9¼	10	¾	10¼	10	¾	⅜	⅜	4560	16500	54000	4875	22400
7	.31	12½	7⅞	8⅝	1.1	⅜	⅝	2⅛	6.8	1	9	9⅝	12	¾	10⅞	12	¾	⅜	⅜	4465	15900	52600	4800	23500
7½	.324	13½	8⅜	9⅛	1.14	⅜	⅝	2⅜	6.6	1	9⅞	10¼	12	¾	11⅝	12	⅞	⅞	⅜	4375	17800	42400	4025	23600
8	.337	14	8⅞	9⅝	1.	⅜	⅝	2¼	6.34	1	10⅝	11	12	⅞	12⅛	12	⅞	⅞	⅜	4297	20300	48400	3935	25600
8½	.35	14⅝	9⅜	10½	1.00	⅜	⅝	2⅜	6.6	1	10¾	11⅝	12	⅞	12¾	12	⅞	⅞	⅜	4225	22300	53300	4550	24400
9	.363	15¼	9⅞	11⅛	1.00	⅜	⅝	2⅝	6.5	1	11¼	12¼	12	1	13¼	12	1	½	⅜	4167	24600	44900	3990	22500
9½	.377	15⅞	10⅜	11⅞	1.00	⅜	⅝	2½	6.6	1	11⅝	12¾	12	1	13⅞	12	1	½	⅜	3950	25900	47100	4175	23200
10	.39	16⅝	10⅞	12⅛	1.04	⅜	⅝	2⅝	6.6	1	12⅛	13¼	12	1	14½	12	1	½	⅜	3910	28200	51300	4220	23500
10½	.403	17⅛	11⅜	12⅝	1.06	⅜	⅝	2⅝	6.5	1	13⅛	14¼	12	1	15⅛	14	1	½	⅜	3875	26300	48000	4240	23600
11	.417	17⅝	11⅞	13¼	1.	⅜	⅝	2 11/16	6.4	1	13⅝	14¾	12	1	15⅝	14	1	½	⅜	3840	28400	51800	4420	24600
11½	.43	18¼	12⅜	13⅝	1.	⅜	⅝	2 13/16	6.4	1	14⅛	15¼	16	1	16¼	16	1	⅝	⅜	3800	26800	48500	4320	23500
12	.443	18⅞	12⅝	14⅜	1.01	⅜	⅝	2⅞	6.4	1	14⅝	15⅝	16	1	16⅞	16	1	⅝	⅜	3780	29100	53000	4500	22000
13	.47	20	14	15½	1.	⅜	⅝	3	6.4	1	15⅞	16⅞	18	1	18	18	1	⅝	⅜	3600	28800	52400	4600	22300
14	.496	21⅛	15⅜	16⅝	1	⅜	⅝	3⅛	6.25	1	17⅛	18¼	18	1	19¼	18	1	⅝	⅜	3570	33300	48100	4050	22500
15	.525	22⅜	16⅝	17⅞	1.	⅜	⅝	3⅜	6.1	1	18⅛	19¼	20	1	20½	18	1	⅝	⅜	3540	37100	53600	4460	23200
16	.549	23⅝	17⅞	19⅛	1.	⅜	⅝	3⅝	6.2	1	19⅜	20½	20	1	21⅝	20	1	⅝	⅜	3420	37100	53500	4525	22400
17	.576	24½	18⅛	20	1.	½	⅝	3½	6	1	20⅛	21¼	20	1	22⅝	20	1	⅝	⅜	3400	40900	45800	3930	23800
18	.602	25½	19⅛	21⅛	1.	½	⅝	3⅝	6	1	21	22⅛	22	1	23⅝	22	1	⅝	⅜	3384	42000	47000	3970	24600
19	.629	26⅝	20⅛	22¼	1.	½	⅝	3¾	6	1	22⅛	23¼	22	1	24⅝	22	1	⅝	⅜	3350	45200	50600	4395	23000
20	.655	27⅝	21⅛	23⅜	1.	½	⅝	3⅞	6	1	23⅛	24¼	24	1	25⅝	24	1	⅝	⅜	3280	48700	51200	4250	22800

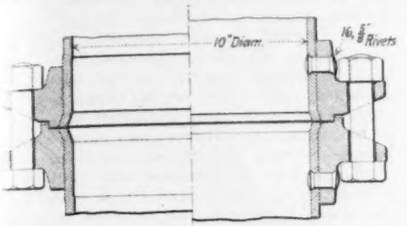


Lovekin Flange Applied to 12-in. Pipe, Showing Male-and-Female Joint for Pressures Over 175 lbs., and Plain Joint for Pressures Under 175 lbs. per. sq. in.

FIG. 1. PROPOSED STANDARD STEAM PIPE FLANGE FOR MARINE WORK, FOR PRESSURES UP TO 300 LBS. PER SQ. IN. Designed by L. D. Lovekin.

...arily? I have been informed by parties well versed ...his line that they are ordering these ponderous steel ...ank for safety only.

...th restrictions upon weight such as are imposed by ... Engineering Department of the U. S. Navy, it be... of interest to all to see that there is no unneces... excess of weight carried on ship-board, such as ...ld necessarily follow were we to attempt the use ...the bulky and yet weak flange list known as the ...chanical Engineers' Standard," which, in most cases ...er my supervision, has proven so inefficient in the ...ber of bolts used as to often require the placing



Expanded Connection as Proposed for Standard.

Single-Riveted Connection as Used by U. S. Navy on Cruiser "Washington" and Battleship "Kansas."

Fig. 2. Lovekin Flange as Used with Single Riveted Connection of U. S. Navy.

of additional bolts between these multiples of four after the work has been erected. Personally, I do not believe there is a shipbuilder or marine engineer in the United States who would support such a standard as is used in the general commercial work for pressures above 150 lbs. per sq. in.

In my opinion, any standard flange list should have the joints or flanges equal at least to the bursting strength of the pipe itself, so that we could rest assured that, in the case of water hammers and other shocks of a similar nature we had at least done our best from an engineering standpoint.

The writer having given his best attention to the subject of pipe flanges for several years, and constantly observing the various troubles that were brought to his attention, begs to submit to the marine engineering world a series of flanges which have been designed very carefully, with a view of overcoming all possible objection, and will be very glad to hear from any of our engineers relative to the same. It might be well to state that my reason for calling particular attention to the marine engineering world in this direction is due to the lack of any one standard which covers both the Navy Department and the merchant marine.

With this in view, I present herewith a full list of flanges, Fig. 1 and the accompanying table, from 2 ins. to 20 ins. diameter, designed for a working pressure of 300 lbs. per sq. in., which I have termed "A Proposed Universal Standard Flange List." The intention is to use the list as designed for pressures between 200 and 300 lbs., and then to modify the size of the bolts used for pressures between 100 and 200 lbs. on all pipes above 4 ins. diameter, so that we would never use less than a ⅝-in. bolt within the range of sizes given; the same diameter of flange, diameter of pitch circle, and number of bolts, would be used on all pressures between 100 and 300 lbs.

It has been my aim to produce a list whereby various methods may be employed for attaching flanges to pipes, without affecting in any manner the principal points of advantage sought for. In order to fully explain my meaning in this direction, sections of 10-in. pipe with flanges attached by various methods are shown in Figs. 2, 3 and 4. It will be noticed that I have made a comparison of the single-riveted flange with the pipe flared at the outer end to an angle of 20°, Fig. 2. I have also shown the double riveting, such as we have used on all our merchant ships, with the pipe calked inside and outside, similar to that of a boiler-shell joint, Fig. 3. I also show the double-riveted joint as used by the U. S. Navy, wherein the end of the pipe is pened over into a recess in the face of the flange, Fig. 4.

In all these designs I have kept the section where the bolt passes through constant throughout. Knowing this section to be more in accordance with mechanical principles than the parallel section commonly employed, caused me to adopt this form, and I have had numerous samples made and tested, so as to prove my statements.

Where the expanded joint of Fig. 1 is used I am certain (from tests) that, if necessary, we can burst the pipe without in any way affecting the expanded joint when properly made. Furthermore, the expanded joint is at least 25% stronger than the average form of riveted joint, such as shown in the accompanying prints.

It is also absolutely impossible to have a leak between the flange and the pipe when so expanded, for I have made numerous tests wherein I have heated both the pipe and the flange red hot, and then plunged the same in water, and afterwards placed them under a test pressure of 5,000 lbs. per sq. in. without any sign of leakage whatever. The reason for this will be apparent to all when it is considered that the ends of the pipes themselves form the joint and any additional clamping effect produced by the bolts has a tendency to pull the pipe closer to the flange, in direct proportion to the angle of flare at the end of pipe, the small shoulder at the end preventing the pipe being forced out of the flange. As stated previously, my aim has been to produce a flange list whereby any of the methods shown may be employed without affecting the principal points of advantage sought for.

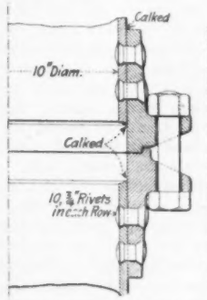


Fig. 3. Lovekin Flange as Used With Double-Riveted Connection of Merchant Marine Practice.

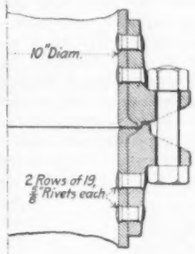


Fig. 4. Lovekin Flange as Used with Double-Riveted Joint of U. S. Navy.

In order that there should be no one particular form of flange connection considered in this matter I have, as before stated, considered all of the joints used in marine work at the present time, with the exception of the welded flange. My reason for not considering this is on account of the enormous expense necessary to make the same, and also the inability of using a flange welded to the pipe to greatest advantage, on account of the very small diameter of flange and also the ability to use a small pitch circle, due to not having any boss on the back of the flange; this type of joint could never be employed as a universal standard, inasmuch as any standard flange list adopted must have the following characteristics, viz.:

We should be able to join a pipe to a valve or similar casting, when made of cast iron, the thickness of metal in such valves or castings being always in excess of the thickness of material in the pipe itself; we should also provide for pipes having their flanges riveted to the pipe. Consequently we must have a pitch circle that will readily conform to the above, so that the nuts or heads of the bolts may clear the fillet in the casting. All flanges submitted by me have had these points fully considered, and the advantages sought are as follows:

(1) A uniform outside diameter of pipe flange, for pressures between 100 and 300 lbs.

(2) A uniform pitch circle for bolting flanges together.

(3) A sufficient number of bolts to be equal in strength to the bursting pressure of the pipe.

(4) A flange section fully as strong as the bolt section in all cases, so designed that the weight of the flange is reduced to a minimum.

(5) To have all bolts entirely outside of the gasket, thereby avoiding troublesome leakage through bolt holes, such as is often encountered.

(6) That the areas of the gaskets between flanges may be subjected to a uniform pressure per square inch in all cases by means of the number of bolts used, said total pressure on gaskets being in excess of the tendency to pull joints apart (due to the internal pressure in the pipe) without subjecting the bolts to a stress of over 5,000 lbs. per sq. in. under working pressure. This feature of the gasket area is a very important one, and the design submitted requires the minimum amount of material that could be desired for such purpose, thus securing a great saving to both the builder and owners in the supply and renewal of these gaskets.

(7) The flanges, as shown, may be used for either copper or steel pipe, and can be made of steel or iron for copper pipe when not subjected to corrosion (such as pipes in the bilges of ships or in any other locality where galvanic action plays an important part), therefore eliminating the practice of brazing the copper pipe to the flange, which, from results shown, is positively not a necessity, but rather a source of danger. Where necessary or advisable to use brazing metal or bronze flanges, they may be of the same sectional area as the steel flanges shown, inasmuch as these steel flanges have been designed for steel pipe, and the same sections if made of brazing metal for copper pipe will bear practically the same relation to the pipe as the steel flanges to the steel pipe.

The third and fourth desiderata are the ones I have given great consideration, and are the basis of all my work. In all of my work I have taken the bursting strength of the pipe as the basis for all calculations relating to these points. The stresses in the flange and the bolts, as calculated in detail for each size of flange, are given in connection with the dimensions, in the table accompanying Fig. 1.

It will be noted that for pressures above 175 lbs. I have used a male and female type of joint; for pressures below 175 lbs. a plain joint is used, Fig. 1.

While, as stated before, any of our well-known methods may be employed in connection with this universal standard flange list, I would state that the advantages of the expanded pipes as shown are as follows: No heating, no brazing, no riveting, no leakage such as is often encountered through rivet holes or between pipe and the inside of flange, no crystallization. Joint may be designed to exceed the strength of the pipe. By the omission of rivet holes the pipe is about 25% stronger than riveted pipe. Furthermore, I have made many samples wherein from actual test the pipe showed an increase in strength due to the cold rolling it received of about 15%.

In conclusion, I would state that with a flange constructed on this principle the tightness of joints at the gasket will not be affected by the pitch of the bolts, inasmuch as the great depth of flange over the gasket is not susceptible of any deflection whatever, and it is therefore impossible to blow a gasket out, due to the deficiencies such as occur in the older forms of flange connections. All that is needed in a design of the character shown is a sufficient number of bolts to enable the flanges to be drawn together to such an extent as to produce a pressure on the gasket in excess of the tendency to pull apart, and a section of flange that will not be unduly strained under such pressure. The depth of the flange through the bore must be sufficient for either single or double riveting or expanding, and the depth of bores in the expanded joints shown herewith are the results of numerous tests made by me, and are what I consider the least possible for safety.

A comparison of the flanges here proposed with those in use by the U. S. Navy Department, whether the single-riveted type shown in Fig. 2 or the double-riveted type of Fig. 4, shows a great saving of weight by the proposed form, in addition to its great superiority in strength.

THE PROPOSED NEW SEWAGE SETTLING TANKS, CONTACT AND STORM BEDS FOR SHEFFIELD, ENGLAND.

After some years of experimentation on a practical scale the City of Sheffield, England, has decided to construct new sewage disposal works consisting of settling tanks, contact and storm water beds. The scheme has been conditionally approved by the Local Government Board, which has sanctioned a loan of £270,000 for the purpose. The following brief particulars regarding the new works have been taken from the annual report of Mr. C. F. Wike, M. Inst. C. E., City Surveyor, the report covering the year ending March 25, 1905.

The existing sewage works were built some years ago. They were designed to treat 12,000,000 U. S. gallons a day by precipitation with lime. They are now inadequate, on account of increased population, increased trade wastes, stronger sewage and higher requirements of purification than formerly existed. During the year 1904-5 a total of nearly 6,300,000,000 U. S. gallons, or some 17,000,000 U. S. gallons a day, of sewage were treated by precipitation and about 400,000,000, or 1,100,000 U. S. gallons, a day, by bacterial processes. Some of the bacteria beds have been in operation six years. Tests of sedimentation and of septic tanks have also been made.

The new works sanctioned by the Local Government Board include settling tanks, contact beds, storm beds and various accessories, and the loan is also to cover the cost of land for extensions. Postponement of an expenditure of £100,000 for second contact beds has been allowed by the Board and the city hopes that experience will show that second contact is unnecessary.

In common with most large British sewage works Sheffield proposes to pass the sewage through screens of various meshes, and into catch pits before it goes to the settling tanks. Two old catch pits, about 42 × 14 ft. in plan, will be modified and used and two new ones will be constructed, each 22½ ft. in one dimension and 31 and 37 ft., respectively, in the other. From each catch pit a conduit will lead to a channel 22 ft. wide connecting with a main feed channel 20 ft. wide, from which branch channels 16 ft. wide will extend to the settling tanks.

There will be nine new settling tanks, of an average size of 221 × 91 ft., and a capacity of about 1,200,000 U. S. gallons each. The bottoms

of the tanks will be sloped from the center up to the sides and from the inlet to the outlet end, the fall longitudinally being 1 in 74. A sludge drain 18 ins. in diameter is to be laid down the center of each tank and will connect with a main drain laid beneath the feed channels. The tank effluent will be discharged over weirs. There will be two submerged walls and four scum boards across each tank; one wall 14 ft. from the inlet end and the other 18½ ft. from the outlet end. The tank walls will be built of brindled brick in cement, and the bottom, sludge drains and culverts will be of concrete.

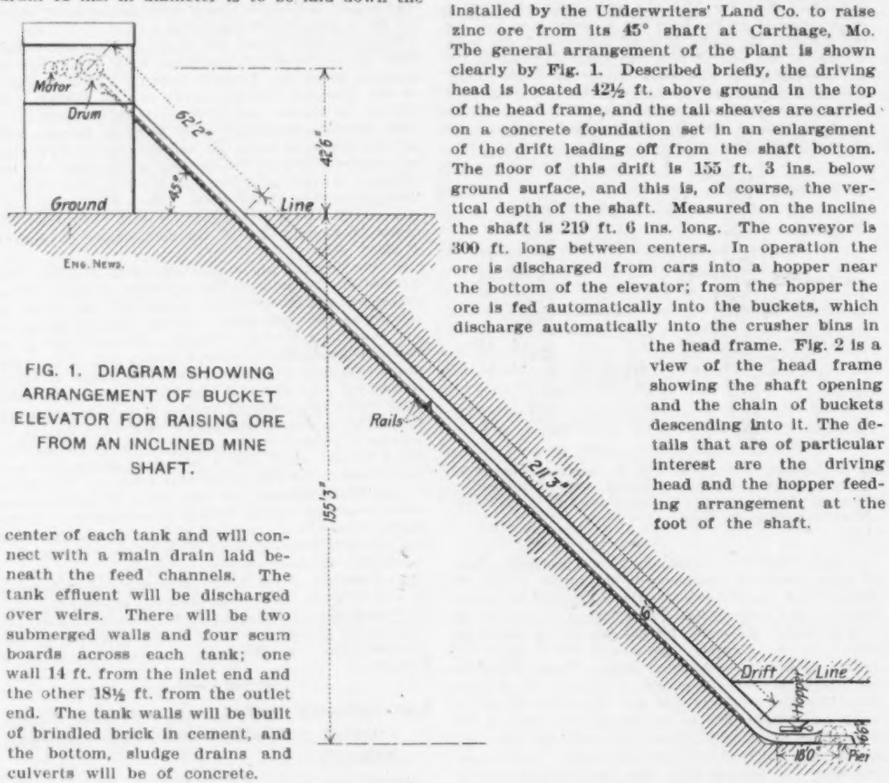


FIG. 1. DIAGRAM SHOWING ARRANGEMENT OF BUCKET ELEVATOR FOR RAISING ORE FROM AN INCLINED MINE SHAFT.

When the new tanks are in working order the old chemical precipitation tanks will be rearranged to provide seven sedimentation tanks with a combined capacity of about 7,200,000 U. S. gallons, thus giving a total tank capacity of 19,200,000 U. S. gallons.

The settling tank effluent will flow through a main channel 20 ft. wide to the single contact beds and to the storm beds, passing under a river between the two areas by means of an inverted siphon. Eight channels 6 ft. wide and two channels 10 ft. wide branch off to the single contact beds and to the storm beds, respectively. Supply pipes 2 ft. in diameter will extend from the lower ends of these conduits to and along the bottom of each bed to or near its center, there connecting with a distributing reservoir or chamber, 13½ ft. in diameter in the contact beds and two chambers each 10½ ft. for the storm beds.

The 60 new contact beds of about ½ acre each will have concrete walls and floors, 4 ft. of contact medium (kind not stated) and 6-in. open jointed land drain pipes, embedded half their depth in the concrete floor. These sub-drains will connect with main drains leading to outlet chambers, two for each bed, controlled by valves. Besides the 30 acres of new beds about 1½ acres of old beds will be available.

The sewage is to be destributed over both the contact and the storm beds by grips or channels formed in the bed medium.

The 16 storm beds of about one acre each have 3 ft. of filtering medium and their distributing chambers are fitted with valves designed to permit the use of the beds as contact as well as storm beds. The original plans for the storm beds provided for placing the medium directly on the natural ground, but the Local Government Board demands concrete in place of natural bottoms.

At the outset, the sludge from the settling tanks will be disposed of by the lagoon system.

The old precipitation works occupied 23 acres, to which 90 acres have been added by recent purchase.

A BUCKET ELEVATOR INSTALLATION FOR A 155-FT. ZINC MINE SHAFT.

A continuous bucket elevator has recently been installed by the Underwriters' Land Co. to raise zinc ore from its 45° shaft at Carthage, Mo. The general arrangement of the plant is shown clearly by Fig. 1. Described briefly, the driving head is located 42½ ft. above ground in the top of the head frame, and the tail sheaves are carried on a concrete foundation set in an enlargement of the drift leading off from the shaft bottom. The floor of this drift is 155 ft. 3 ins. below ground surface, and this is, of course, the vertical depth of the shaft. Measured on the incline the shaft is 219 ft. 6 ins. long. The conveyor is 300 ft. long between centers. In operation the ore is discharged from cars into a hopper near the bottom of the elevator; from the hopper the ore is fed automatically into the buckets, which discharge automatically into the crusher bins in the head frame. Fig. 2 is a view of the head frame showing the shaft opening and the chain of buckets descending into it. The details that are of particular interest are the driving head and the hopper feeding arrangement at the foot of the shaft.

The stands and track rails which carry the continuous chain of buckets are shown by Fig. 3. In the shaft the stands are supported on concrete piers, and above ground they are carried by a timber trestle. The buckets are of the overlapping type, are made of malleable iron and are 24 ins. long. They are carried by two strands of drop forged steel chain, with 1½-in. steel axles to which are fastened chilled, dust proof, oil chambered track wheels 6 ins. in diameter. The track consists of heavy T-rails arranged as shown by Fig. 3. The chain travels 50 ft. per minute.

Details of the driving mechanism are shown by Figs. 4 and 5. Fig. 4 gives the general arrangement. The motor is of the alternating current type. Practically the only feature that calls for special notice is the ball brake or friction ratchet designed to prevent the elevator from running backward in case the power is cut off. This device operates on the large spur wheel on the driving shaft, and as will readily be seen is designed to allow free rotation forward of the spur but to grip the spur wheel and hold it fast upon its making any backward movement. The details of the ball brake or ratchet are shown to large scale by Fig. 5. They call for no explanation except to note that the balls a... spectively 2 ins., 1½ ins. and 1 in. in diame...

The tail end mechanism of the eleva... shown by the drawings of Fig. 6. The f... of particular interest is the automatic fe... arrangement. It is in the form of a pan ... entirely covers the bottom of the hopper ... is mounted on four wheels and axles of the ... size as are used for the buckets. This pa... operated by means of a crank located on a... mounted shaft, which is driven through ch... and sprocket wheels from the elevator s... As each bucket passes under the pan, the ... moves forward a certain distance and th... drawn back; this allows a certain amount o... to be deposited directly into the bucket. ... movement of the pan can be varied to sui... quirements by changing the length of the th... of the crank; the variation may range anyw... between nothing and 100 tons per hour.

The elevator as described was built and ... stalled by the C. O. Bartlett & Snow Co., ... Cleveland, O., and we are indebted to them for the information from which this description has been prepared. The price of this elevator was $3,700 f. o. b. Carthage, Mo. This did not include the cost of erection.

FIG. 2. VIEW OF HEAD FRAME AND BUCKET ELEVATOR AT UNDERWRITERS' LAND CO.'S ZINC MINE, CARTHAGE, MO.

REDUCING THE VOIDS IN BROKEN STONE FOR CONCRETE BY MIXING DIFFERENT SIZES.

By E. J. McCaustland.*

Much has been written on the "theory of concrete," and numerous suggestions have been made as to the best method of proportioning the ingredients in order to secure a maximum of strength at minimum cost.

Whatever method may be suggested or adopted, it will, after all, be found to reduce to a process of determining the voids in the broken stone or gravel and proportioning the mortar in slight excess of these voids. It will, in general, be found desirable to reduce the proportion of voids in the

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aggregate as far as possible without materially adding to the cost. This will save cement without reducing the strength of the product, since, if the voids in the aggregate are entirely filled, the strength of the matrix determines the strength

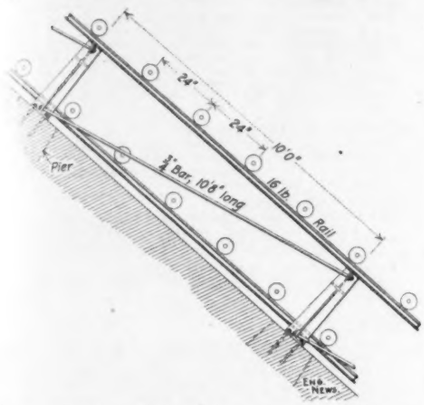

Fig. 3. Tracks and Track Supports for Buckets and Chain.

of the whole. A stone having 40% voids will, under like conditions, make a concrete fully as strong as a stone having 50% voids, and will accomplish this result with 20% less mortar. Therefore any information as to sizes of broken stone or combinations of sizes with minimum proportion of voids, should be welcomed as offering possible opportunity for economy in cost with no sacrifice of strength or efficiency.

The following results of tests on mixtures of stones of various sizes are reported as throwing some light on the question of minimum voids in broken stone. The tests were begun under the direction of the writer by Mr. W. H. Gerwig, of the class of 1905 of Cornell University. Mr. Gerwig's results have been verified, and additional tests made by the writer, the results of which are included in the following tables.

The stone experimented on was crushed argillaceous limestone which came from the crusher in a fairly regular angular form and with only a small proportion of flat or spally pieces. The stone was separated by screens into sizes as indicated in the following tabulation. Each size is designated by the screen which passed it, the range in dimension being down to the screen on which it was held. Thus 1-in. stone includes all material passing a 1-in. screen and held on ½-in. screen, etc.

The voids were determined in the following manner: The stone to be experimented with was first thoroughly wet to provide for absorption, and then, after draining off the water, a tight box of 1 cu. ft. capacity was filled with the stone and moderately rammed. Water was then carefully poured into the box until all the interstices in the stone were filled. The volume of water necessary to be used represented the voids in 1 cu. ft. of stone. The mixtures of stone of two or three sizes were proportioned and thoroughly mixed before being placed and tamped in the measuring box. To insure uniformity of the mixing in all cases was a matter of some difficulty, and to determine when this object was finally accomplished required very careful observation.

In general, mixtures containing large proportions of the larger sizes of stone, were seemingly more difficult to thoroughly incorporate than were those containing a less proportion of the larger sizes, or mixtures made up wholly from a combination of smaller sizes. For instance, ½-in. and 2-in. sizes mix fairly well in all proportions;

Table I.—Sizes and Voids in Stone Used in the Experiments

Stone Passing Screen of Opening	Stone Held on Screen of Opening	Referred to as Size of	Per Cent. Voids
1/2-in.	1/8-in.	1/2-in.	38.5
1 "	1/2 "	1 "	41.5
1 1/2 "	1 "	1 1/2 "	45
2 "	1 1/2 "	2 "	46
2 1/2 "	2 "	2 1/2 "	47

TABLE II.—SHOWING MIXTURES OF TWO SIZES OF STONE.

Proportional Parts of Sizes. 1/2-in.	1-in.	1 1/2-in.	2-in.	2 1/2-in.	Ratio of Dimensions.	Per Cent. of Voids.	Remarks.
1/3	2/3	....	....	....	1 : 2	37	Easy to make uniform mixtures of these sizes.
2/3	1/3	....	....	....	1 : 2	34.5	
1/2	1/2	....	....	....	1 : 2	37.5	
....	1/4	....	3/4	....	1 : 2	42	
....	3/4	....	1/4	....	1 : 2	39.5	
....	1/3	....	2/3	....	1 : 2	41	
....	2/3	....	1/3	....	1 : 2	40.5	
....	1/2	....	1/2	....	1 : 2	40	
....	1/4	3/4	....	....	2 : 3	41.5	
....	3/4	1/4	....	....	2 : 3	40	
....	1/3	2/3	....	....	2 : 3	45	
....	2/3	1/3	....	....	2 : 3	39	
....	1/2	1/2	....	....	2 : 3	41.5	
....	1/3	....	....	2/3	2 : 5	37.5	Not easy to make uniform mixture.
....	2/3	....	....	1/3	2 : 5	37.5	
....	1/2	....	....	1/2	2 : 5	37.5	
....	....	1/3	2/3	....	3 : 4	44	
....	....	2/3	1/3	....	3 : 4	43.5	
....	....	1/2	1/2	....	3 : 4	44	
....	....	1/3	....	2/3	3 : 5	42	
....	....	2/3	....	1/3	3 : 5	42	
1/3	....	....	2/3	....	1 : 4	38.4	These mixtures all easily made.
2/3	....	....	1/3	....	1 : 4	36.2	
1/2	....	....	1/2	....	1 : 4	36.3	

TABLE III.—SHOWING MIXTURES OF THREE SIZES OF STONE.

Proportional Parts of Sizes 1/2-in.	1-in.	1 1/2-in	2-in.	2 1/2-in.	Ratio of Dimensions.	Per Cent. of Voids.	Remarks.
1/6	1/6	2/3	....	....	1 : 2 : 3	38	
1/3	1/3	1/3	....	....	1 : 2 : 3	36	
1/4	1/4	1/2	....	....	1 : 2 : 3	34	Easy to mix uniformly.
1/8	1/8	3/4	....	....	1 : 2 : 3	39	
3/8	3/8	1/4	....	....	1 : 2 : 3	38.5	
1/6	1/6	....	2/3	....	1 : 2 : 4	35	
1/3	1/3	....	1/3	....	1 : 2 : 4	33.5	
1/4	1/4	....	1/2	....	1 : 2 : 4	35	
1/8	1/8	....	3/4	....	1 : 2 : 4	39	
3/8	3/8	....	1/4	....	1 : 2 : 4	36.5	
1/6	1/6	....	....	2/3	1 : 2 : 5	33	Difficult to mix uniformly.
1/3	1/3	....	....	1/3	1 : 2 : 5	33.5	Easy to mix uniformly.
1/4	1/4	....	....	1/2	1 : 2 : 5	33.5	" " "

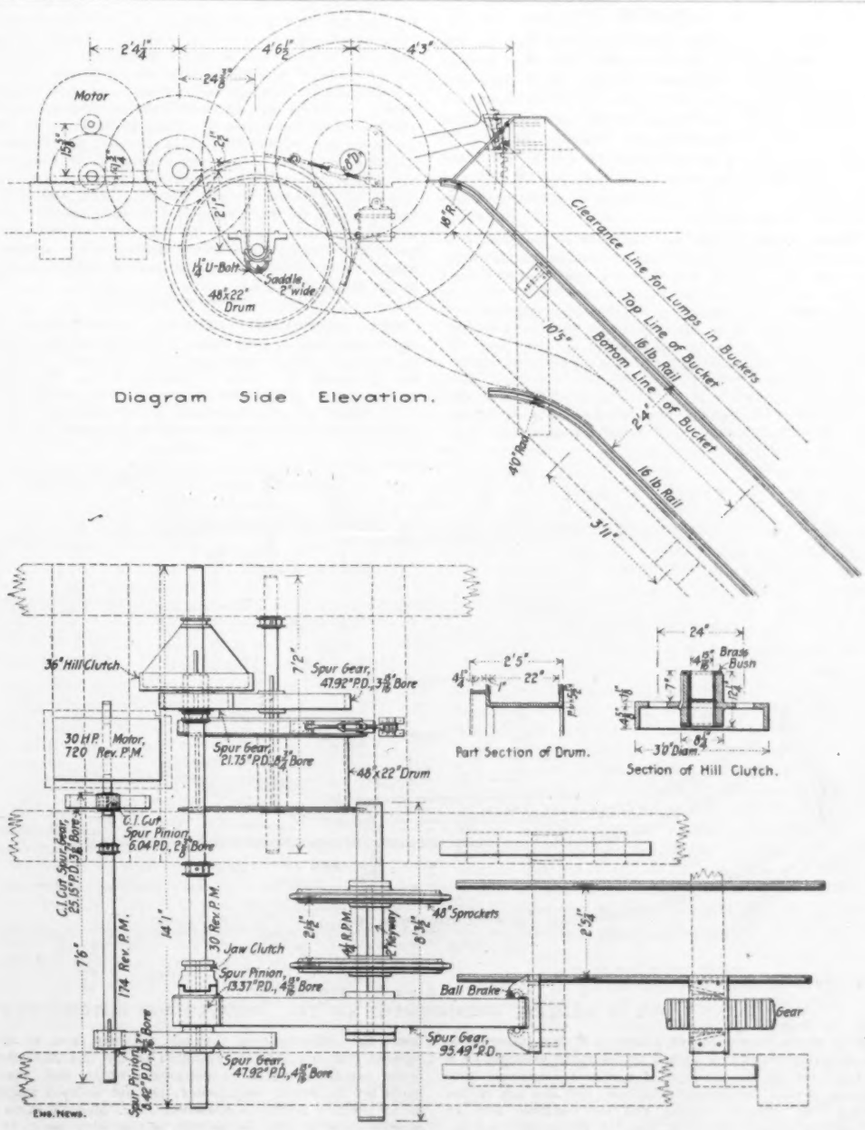
Motor
Clearance Line for Lumps in Buckets
Top Line of Bucket
16 lb. Rail
Bottom Line of Bucket
16 lb. Rail
40' Rail
Diagram Side Elevation.
36" Hill Clutch
Spur Gear, 47.92" P.D., 3 15/16" Bore
30 H.P. Motor, 720 Rev. P.M.
Spur Gear, 21.75" P.D., 3 3/16" Bore
48"x22" Drum
C.I. Cut Spur Pinion, 6.04 P.D., 2 7/16" Bore
C.I. Cut Spur Gear, 23.15" P.D., 3 3/16" Bore
30 Rev. P.M.
174 Rev. P.M.
Jaw Clutch
Spur Pinion, 13.37" P.D., 4 15/16" Bore
Spur Pinion, 3.42" P.D., 2" Bore
Spur Gear, 47.92" P.D., 4 15/16" Bore
Ball Brake
Spur Gear, 95.49" P.D.
48" Sprockets
Gear
Part Section of Drum.
Section of Hill Clutch.
Brass Bush
30" Diam.
Plan.

FIG. 4. GENERAL ARRANGEMENT OF ELEVATOR-DRIVING MECHANISM IN HEAD FRAME.

so do ¼-in. and 1-in. sizes; but a uniform mixture of 1-in. and 2½-in. stone is not easy to secure.

The writer believes, however, that these are difficulties attendant upon experimental work only, and that they would not be serious in the event of an attempt to fabricate a concrete either by hand or machine mixing. This view is supported by experience and careful observation dur-

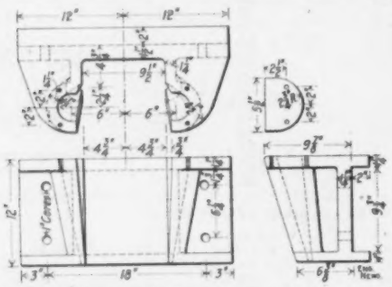

Fig. 5. Details of Ball Brake or Friction Ratchet.

ing the construction of a large concrete sewer near Chicago,* where no particular difficulty was encountered in machine mixing of sizes, which ranged from ⅛-in. to 2½-ins.

AMERICAN RAILWAY TRACK FROM A FRENCH POINT OF VIEW.

Among the foreign engineers who visited this country last year in connection with the International Railway Congress, was Mr. M. G. Bauchal, Assistant Engineer of Track and Works of the Western Railway of France. In a recent number of the "Revue Generale des Chemins de Fer" he has presented a short paper reviewing his ideas on American railway track construction and equipment, and we give below a condensed abstract of this paper:

While it may be true that the track of American railways has been materially improved within recent years, it must still be said that they are now in general of a very second-rate character. This condition, which at first seems astonishing in so rich and progressive a country, seems to be due to the fact that the people regard progress as of first importance for their vast territories, and are impressed with the necessity of expending only what is absolutely necessary for each enterprise and to reserve as much as possible for other new enterprises. The consideration of the safety of individuals is subordinated to that of the benefit of the mass. While we in Europe regard safety as of first importance, in America they think rather of industrial progress as a whole. While the track is of mediocre character, it is strong enough for general purposes, and the use of the truck, the central coupling and the vestibule help to make the trains ride easily even on this track. As the country develops and traffic increases, the track is improved; more ballast is laid, and the rails are replaced with larger and heavier rails. The ties are in good condition and very numerous, and it would seem to be only necessary to use screw spikes and to ensure a regular uniform bearing to ensure good track.

We see, then, that the Americans have acted advisedly in seeking for cheapness of first cost of construction. Perhaps this has been the cause of serious accidents, but without it the railways would not have had their marvellous development. In the construction of our main lines in the colonies, one may follow this example in future and avoid a too costly perfection in first construction. We may, as in America, make initial economies in substructure and track, and, on the other hand, introduce a high class of rolling stock.

As to rail joints, if a life of 50 years is assumed for a steel rail, there is no joint in America or elsewhere which will prevent for such a period the wear and deprivation of the ends of the rails. But there are several joints which will give good service for 15 to 20 years. The experience with the practice followed (at my suggestion) by the Western Ry., of cutting off the worn ends of the rails, shows that this is easy and inexpensive. With two operations of this kind, good joints can be had for 50 years, and this would only reduce the length of the rails about 5 ft.

There is a general use of hand cars and motor vehicles for the section men and for track inspection, and it is surprising that these are so little used in France. They would enable the length of sections to be increased on the lines of light traffic, and would greatly reduce the fatigue of long walks over the track. Further, they would largely reduce the use of special inspectors or private cars attached to trains, with the enormous loss of time due to stops at stations. On the Western Ry., three-wheel and four-wheel hand cars or velocipedes are employed only on lines of minor importance. The more extended use of such machines is very desirable.

In operating switches and signals at interlocking plants, electricity and the high and low pressure pneumatic systems are employed. Some of these are in use in Paris, but we have others in Europe; the Bianchi-Servettaz hydraulic system and the Carpentier Ducousso and Rodary systems in France, and the Siemens & Halske system in Germany. We are, however, rather behind the United States in the development of such plants. The use of electricity seems to be the most advantageous, as it alone meets the various conditions and requirements, while it must in fact be used as complementary in the other systems.

*See paper, "Heavy Sewer Construction in Concrete," Trans. Assoc. Civ. Eng. of Cornell University, Vol. X., 1902.

THE ERIE-GREAT CENTRAL CONSOLIDATION which was noted in these columns a short time ago seems to have met serious obstacles. The Cincinnati, Hamilton & Dayton Ry., with its subsidiary line, the Pere Marquette Ry., was bought by the Erie Ry. through Mr. J. P. Morgan as an intermediary, a short time ago. Recently an inspection trip was made by a number of prominent Erie officials over the new property, and it developed that the physical condition of the line and also some extensive traffic arrangements which had been made by the former management are not in such shape as to promise profitable operation. Mr. Morgan subsequently made an offer to the Erie to take over the C. H. & D. (or Great Central) system, relieving the Erie of any obligations on the account of its purchase. This offer has been accepted by the Erie.

ENGINEERING MATERIAL PRICES.

TRACK FASTENINGS—
Pittsburg f. o. b. cars—Angle bars, 1.75 cts.; [illegible] 2 to 2.05 cts.; bolts, square nuts, 2.15 cts.; hexagonal nuts, 2.40 cts.

RAILS, STEEL—
New York—at mill—New standard sections, $28; ard, second quality, $27; light sections, 12 lbs., $24 to $30; old iron, f. o. b. New York, $2 $23; old steel, long lengths, $16.50 to $17; old short pieces, $16.50 to $17; relayers, $25 to $2
Pittsburg—Standard sections, at mill, $28; st second quality, $27; light rails, 12 to 25 lbs., $30.

STRUCTURAL MATERIALS—
New York—at tidewater—Beams and channels, up in., 1.84½ cts.; over 15-in., 1.84½ cts.; angles, and up to 6 × 6, 1.84½ cts.; zees and tees, 1.89 steel bars, half extras, 1.74½ cts.

PLATES AND SHEETS—
New York—at tidewater—Sheared plates, tank, to 1.82½ cts.; flange, 1.84½ to 1.94½ cts.; 1.94½ cts.; still bottoms, 2.04½ cts.; fire box, to 3.05 cts.
Pittsburg—Tank, ¼-in. thick and heavier, 1.60 black sheets, 27 gage, 2.25 cts.; black sheets, 2.35 cts.

IRON—
Pig Iron:
New York—tidewater—Northern; No. 1 X Foundry, Northern, $18.50 to $19.50; No. 2 X Foundry, Northern, $17.75 to $18.75; No. 2 Plain Foundry, $17.25 to $17.75; Gray Forge, Northern, $16.50 to $17; Basic, Northern, $18 to $18.25; Basic, Alabama, $18.75; Basic, Virginia, $19.10 to $19.60; No. 1 Virginia Foundry, $18.60 to $19.10; No. 2 Virginia Foundry, $18.10 to $18.60; No. 1 Foundry, Southern, $18.75; No. 2 Foundry Southern, $18.25; No. 3 Foundry, Southern, $17.50 to $17.75; No. 4 Foundry, Southern, $17 to $17.25; No. 1 Soft, Southern, $18.50 to $18.75; No. 2 Soft, Southern, $18.25; Gray Forge, $16.25 to $16.75; Mottled, $15.75 to $16.25.
Pittsburg—f. o. b. cars—No. 2 Foundry, $18.35 to $18.85; Gray Forge, $17.35; Bessemer Iron, $18.35; Ferro Manganese, $80 to $100; Muck Bars, $29 to $31; Iron Bars, $2.04.
Cast-Iron Pipe:
New York—Carload lots, net tons, 6-in., at tidewater, $27.50.
Chicago—4-in. water pipe, $30; 6-in. and larger, $29, with $1 extra for gas pipe.

MISCELLANEOUS MATERIALS—
Asphaltum:
Ventura and other California asphalts, $20 to $21 per ton at New York; Trinidad refined, $25 per ton; Venezuela asphalt, $25 to $30 per ton; Bermuda asphalt, $25 to $30.
Cement:
Rosendale, in wood, 85 to 95 cts.; Portland domestic, in wood, dock, $1.55 to $1.60; German Portland, $1.90 to $2.10.
Copper:
Lake, 17.75 to 18 cts.; electrolytic, 17.75 to 18 cts.; casting, 17.50 to 17.75 cts.
Lead:
New York, spot, 5.75 cts.
Spelter:
New York, spot, 6.35 to 6.45 cts.
Tin:
New York, spot, 34.85 to 35.10 cts.
Tar:
New York, $5.50 to $6 per bbl.
Pitch:
New York, $3 per bbl.

LUMBER—
New York, wholesale prices:
Piling—Spruce, ordinary cargoes, 6 to 7 cts.; oak, 14-in. butt, 40 to 50 ft., 19 cts.; 50 to 55 ft., 22 cts.; 55 to 60 ft., 23 cts.; 60 ft. and up, 25 cts.; pine, 60 to 65 ft., $8.50; 70 to 75 ft., $10.50; 80 ft. and up, $16.
Railroad ties—Yellow pine, 6 × 9 ins. × 8 ft., 65 to 66 cts.; 6 × 8 ins. × 8 ft., 50 to 61 cts.; 7 × 9 ins. × 8 ft., 71 to 72 cts.; 7 × 8 ins. × 8½ ft., 67 to 68 cts.; 7 × 9 ins. × 8½ ft., 72 to 73 cts.

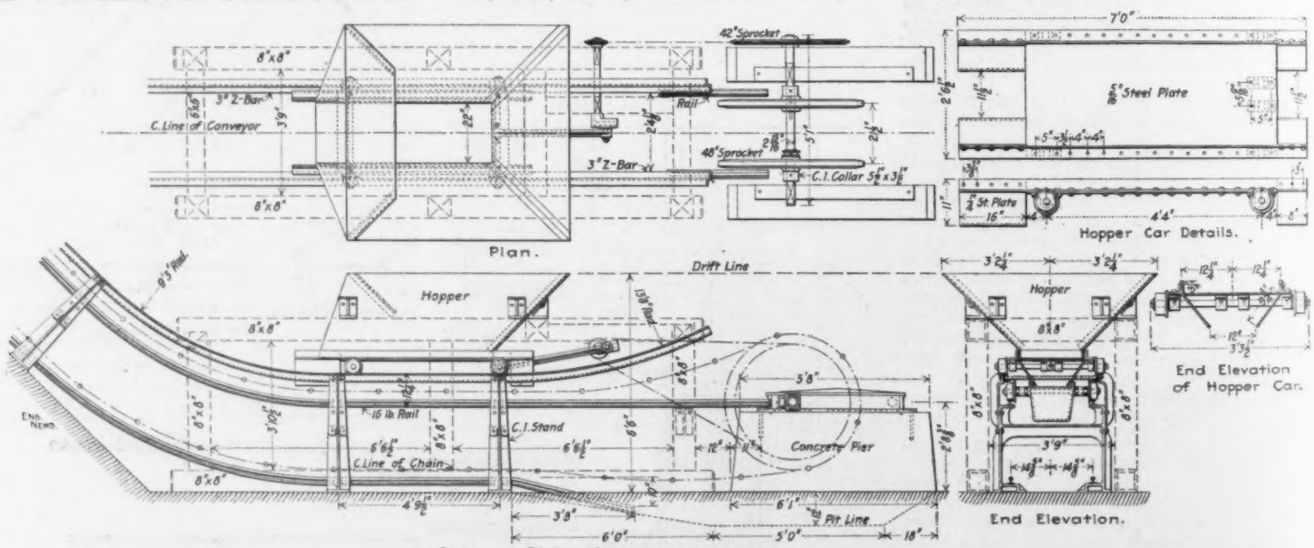

FIG. 6. GENERAL ARRANGEMENT OF TAIL SHEAVES AND AUTOMATIC FEEDING HOPPER IN SHAFT.

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In the columns of this journal we have repeatedly warned investors to be on their guard in accepting claims made for electric iron smelting processes; but the caution seems to need further repetition in view of the large amount of published matter regarding these processes which is being spread abroad in trade journals, and particularly in view of the persistent attempt which is being made to induce the investment of Canadian capital. There are none so blind as those who will not see, and particularly those whose interest it is not to see. The woods are full just now—literally—of all sorts of projects, good, bad and indifferent, for the development of great water-power plants. When the promoter of one of these enterprises is asked what he will do with the power after it is developed, it is very nice to be able to speak glibly of the vast future demand for electric current to smelt iron ore.

If memory rightly serves us, a very strong claim for the electric iron smelting process has been based on the commercial success alleged to have been attained by the Heroult process in Sweden. For the benefit of those who may be interested to learn the truth, we may state that as a matter of fact the Heroult process in Sweden has been anything but a commercial success. On the authority of a well-known American engineer who was financially interested in the experimental plant in Sweden, it may be said that the enterprise was originally started there on the inventor's proposal to utilize Swedish water power for smelting iron ore, and producing steel direct. The furnace when built, however, used scrap iron as a raw material, instead of ore, and the product was of such irregular quality as to be unsalable. At times it contained more sulphur than the raw material charged in the furnace. Better steel has since been made, and some of it has been marketed; but the business was at last accounts, according to our informant, still being conducted at a loss, the steel costing more to make than the price received for it in the market.

Allowances must, of course, be made for the experimental stage of a new enterprise in an unworked field, and there are theoretical reasons why steel made in an electric furnace might be expected to be freer from impurities than steel made with fuel in the ordinary way. But the point we would emphasize is that great caution should be exercised in sifting the claims made for any electric iron or steel-producing process. Allegations of commercial success in European plants should only be accepted when established by an independent investigation. High-grade steel is now produced by well-known and long-practised methods at so low a cost that the electric smelting process, even if made a technical success, is likely to be unable to compete in the market with its well-established rivals.

The Federal Safety-Appliance law limits the variation in height of car couplers to 3 ins., or 1½ ins. above and below the standard height of 33 ins. from the top of the rail. At the October meeting of the Western Railway Club a proposition was brought forward that this limit of variation ought to be increased to 4 or 5 ins., because of the difficulty which is now experienced in keeping cars in repair so that the legal limit of variation will not be exceeded.

Mr. J. J. Hennessy of the Chicago, Milwaukee & St. Paul Ry. enumerated the several causes which affect the height of couplers on an old car loaded as compared with a new car empty as follows:

Wear on journals	¼ in.
Wear on journal brasses	⅜ in.
Wear on wheel treads	¼ in.
Wear on coupler carry-iron	¼ in.
Wear on coupler shank	¼ in.
Wear and set of arch-bars	¾ in.
Permanent set of springs	⅜ in.
Drop of end of car	⅜ in.
Compression of springs	1½ ins.
Total	3⅞ ins.

It will be readily agreed that all these estimates of wear are very moderate and that if a car is permitted to run without adjustment the difference in height of couplers between an old car loaded and a new car empty is quite certain to be more than the legal limit of 3 ins.

The argument made before the Western Railway Club was substantially that with the solid knuckles of the M. C. B. coupler universally replacing the knuckle with two lugs (which had to be used during the transition period while the old link-and-pin drawbars were still in service) the coupler faces in contact are 9 to 10 ins. in length. Thus when two cars come together with a variation of 3 ins. in height there is still 6 to 7 ins. of the coupler faces in contact and it was contended that this might safely be reduced to four or five ins. and still leave the coupler strong enough for safety.

There was strong dissent from this position on the part of one or two members, but a motion was carried that the subject of an increase in the legal permissible variation in height of freight cars be referred to the arbitration committee.

We trust that this committee, which has a high reputation for conservative action on important questions, will see to it that the movement for greater variation in coupler heights receives a quietus. For the safety of the railway service, cars ought to be kept within the present legal limit of variation. Any change in the law to permit greater latitude would be a public misfortune. There is admittedly a little trouble for the car department is fixing old cars, to keep their couplers up to standard height; but this trouble is well worth taking.

The point to which we wish to call special attention is that the safe variation in coupler heights depends on the relative heights *when the cars are running.* It is all very well to say that two cars with 3 ins. difference in the height of their couplers have still 7 ins. width of the coupler knuckle faces in engagement; but run these cars over a rough track and see how wide a face remains in contact on the two couplers. The car with the high coupler may be a new car, empty, and the car with the low coupler an old car loaded and when the new car happens to rise on its springs at the same time that the old car settles down, the width of coupler faces, in contact is reduced quite as far as is safe.

The breaking in two of freight trains has been a fruitful source of accident in the past and while it has generally been due to other parts than the coupler knuckles, it is not desirable that any steps be taken that may create an additional cause for the breaking in two of long trains.

It ought to be remembered, too, that in modern freight train service, with long trains and powerful locomotives, the coupler is subjected to enormous strains. To throw the whole power of a locomotive on the upper or lower five or six inches of a coupler knuckle, or to subject the upper or lower part only of the knuckles to buffing shocks in a railway yard is not good mechanics nor will it tend to economy in the railway service.

The decision to install sedimentation instead of septic tanks for so large a city as Sheffield, England, noted elsewhere in this issue, merits serious consideration by engineers and municipal authorities in this country. No explanation of the grounds for the decision is given by Mr. C. F. Wike, M. Inst. C. E., in the report on which our article is based, but there is evidence that he chose settling tanks deliberately. His decision against septic tanks, as also in favor of contact beds instead of percolating filters, illustrates anew (see editorial notes, Oct. 19, 1905) the fact that no one system or combination of methods of sewage treatment has yet established itself or is likely to establish itself in the minds of engineers as superior to all others, regardless of local conditions. Mr. Wike's decision in favor of plain sedimentation accords with the ideas or inclinations of some American engineers and his choice of contact beds instead of percolating filters indicates that contact beds are by no means dead in England, as some have contended of late. The most notable detail of Mr. Wike's plan is the use of central distributing chambers for the contact and storm beds, with distributing channels or grips formed in the material composing the beds. Similar grips are used at Manchester, England, and elsewhere, but the distributing or outlet chambers are usually if not always located at the corners or sides of the beds, thus making it possible to control at one point the dosing of each of two, three or four beds. It would be interesting to know the relative construction and operation costs of center and corner or side distributing systems and also their relative efficiencies of uniform distribution of sewage over the beds.

The maintenance of rail joints and the improvement of joints when the rail ends have become worn and depressed are two very troublesome problems in the practical work of maintenance of way, and innumerable devices for supporting and strengthening the joints have been used with more or less success. The Western Railway of France has tried a radical method which consists in cutting off the worn ends and drilling new bolt holes, thus practically making new rails of a little shorter length than the old ones. This method was suggested by Mr. Bauchal, Assistant Engineer of Way and Works, who states in a recent paper that experience has shown it to be both easy and inexpensive. He says that if a rail is estimated to have a life of, say, 50 years in main track, there is no joint device that can keep the ends in condition for that time, although there are several devices which might keep them in good condition for 15 or 20 years. But by twice cropping off the worn ends, the rails might be made to give their full estimated life of 50 years. The total reduction in length would be only about 5 ft., (or 15 ins. of cropping at each end for each treatment), or one-twelfth of the length of a 60-ft. rail. A similar method has been used to some extent in this country, and was described in an article in Engineering News of Dec. 27, 1899, which also described a rail sawing and drilling machine used by some large railways for this purpose. About 14 of these portable steam rail sawing and drilling machines are now in use in this country.

Apropos of the current discussion as to the size of Navy which the United States really needs, it is of interest to note that besides the vessels already in commission there are four battleships in the builders' yards on which more than 90 per cent. of the work has been done; three more on which over 80 per cent. of the work has been done, and six more which are one-quarter to three-quarters completed. There are also four huge armored cruisers and two protected cruisers which are over 80 per cent. completed.

These facts are of interest in connection with

Admiral Evans' recent published remark that the Navy has more need now for officers than for ships, and Secretary Bonaparte's reported recommendation that no further appropriations for new battleships be made by the present Congress.

HOW TO DRAW SPECIFICATIONS WHICH WILL SHUT OUT COMPETITION AND MAKE CERTAIN THE AWARD OF THE CONTRACT TO A FAVORED BIDDER.

It is the duty of the engineer or architect responsible for the design and construction of a public work which is to be built by contract, so to frame his specifications that *real* competition may be secured. Any and all requirements are objectionable which tend to debar competent contractors from bidding, which add to the uncertainty and risk of the work or which place the contractor at the mercy of the engineer.

It is easy to frame a defence of such requirements to the effect that they are necessary to provide for such contingencies as arise in the prosecution of the work and to give the engineer proper control to secure the quality of workmanship that is desired. The cold, hard fact is, however, that such requirements have been used and are used as a club to scare off competition and cause the award of the contract to a concern which is "in cahoots" with the powers that be.

There was recently laid before us a specification for a power-house to be built at a State institution in Pennsylvania. The architect of the structure was a close family connection of one of the most prominent of the Philadelphia contractors whose frauds in connection with the filter plant construction were exposed in the report published in our issue of Nov. 2. The Board of Trustees of the institution, however, appear to have been entirely honest; but wholly unacquainted with the devious ways of the up-to-date grafter on public works.

Taking up the specification, the following points may be noted:

First. Bidders were required to deposit $200 in cash with the architect to obtain a copy of plans and specifications costing all told not over $10. This seems like an excessive requirement to ensure return of the plans.

Second. It was specified that the work should commence ten days from the date of notice given by the architect to commence the same, and that the time for completion should date from the time of such notice.

This was the provision which enabled the favored bidders for the Boulevard contracts in Philadelphia to specify a much shorter time than any of the other bidders, as they had it arranged with the Commissioners to withhold notice of the commencement of work for several months.

Under "violation of contract" the architect was given absolute power to notify the contractor in writing, and within 24 hours after the delivery of the notice, the contractor should cease work. The architect had then full authority to immediately purchase anything required, and take possession of all the material on the grounds or built into the work, and the contractor or his bondsman must settle all bills.

Such a provision in any contract would be absolutely null and void in a court of law; but unfortunately many architects and engineers still insert such absurd provisions, and the contractor has no option but to sign what is placed before him or lose the contract.

The payments were specified to be made from "time to time during the progress of the work, but not oftener than once a month," and the architect was given the power to estimate the value of the work, for which 85% would be paid.

We can easily imagine the condition where it would not be convenient for the architect to make an estimate for three or four months, which would leave the contractor in an unenviable position unless he had large financial resources.

Under a clause relating to minor changes, in case any work was not in accordance with the specifications the architect was empowered to accept such work or materials and make such a reduction in the contract price as he might determine, which reduction was to be "final, conclusive and without appeal."

While there were 25 drawings furnished showing most of the details, it was specified that "the contractor should assume all responsibility for any difference that might be found to exist." Such a provision as this, if carried out literally, would require that the contractor should do the work for which the architect is paid, that is, go over the ground thoroughly, remeasure all the quantities and carefully check the drawings and specifications for any errors. The obvious result of such a clause in a specification, even if inserted with honest intent, will be to cause contractors to add a contingent fund to cover errors which may exist, and for which they may be held responsible.

Under the clause relating to extra work there was this unique provision:

> the contractor hereby agrees to furnish such materials and perform such labor as extra work and agrees to accept in full payment thereof a price which shall be fixed by the architect and the Board of Trustees previous to its commencement.

Such a power vested in the architect and Board would be sufficient to bankrupt any contractor should such a proceeding seem desirable. The contractor would have no opportunity for defence, as it is distinctly specified that he shall do such work and accept such payment as they may see fit to offer him.

There was further the too common blanket clause covering anything which the architect might have forgotten reading as follows:

> although the same may not be directly or indirectly so specifically noted on the plans and specifications, the contractor shall execute the same without charge or claim therefor.

One naturally wonders how the contractor can know what will be required unless it is either noted on the plans or covered in the specifications. As before he simply adds a percentage to cover possible demands of the architect.

Under the clause covering the site for the building it was specified that:

> the only land which will be furnished by the trustees to be used by the contractor in the construction of the work shall be that portion upon which the new work is built.

This would preclude the storage of any materials around the building or the use of any driveways. How any contractor could put up a building with only the land on which he is to build provided, it is difficult to see. This was the very clause which when recently enforced against a contractor in Philadelphia, caused him so much annoyance and expense that he was forced into bankruptcy.

There are other features of these specifications well deserving comment; but the above will be sufficient to show the power which the architect held over the contractor and the danger to any contractor who might bid on the work if the architect were not an honest and honorable man.

As stated above, the Board of Trustees in charge of the work were apparently honest men. Some of the contractors who desired to bid, therefore, laid before the members of the Board the difficulty they were under in safely submitting a moderate and reasonable estimate for the work. As a result we are authoritatively informed that the Board of Trustees when it met on the date set for opening the bids decided to reject all the proposals and readvertise the work, stating that the specifications would be altered so that all bidders would have a fair and equal chance, which they did not think was the case at the first bidding.

LETTERS TO THE EDITOR.

The Hauling Power of a Geared Locomotive.

Sir: I read with interest the amusing experiences of a Civil Engineer, related in a "Letters to the Editor," in your issue of Nov. 9.

The writer once had occasion to review some work done by a Civil Engineer (?), included in which was a specification for a geared locomotive to operate over a mountain road. After the clauses stating the maximum curve and grade it was to operate over, came the load the locomotive was to pull and this is what was specified, "The said locomotive is to pull all she can."

Yours truly, A. C. Barrow, C. E.

Yamacraw, Ky., Nov. 28, 1905.

Switch Leads for Narrow Gage Track.

Sir: Can you or some of the readers of the Ne[illegible] me information in regard to the proper length of [illegible]h lead to be used with a three-foot gage? The th[illegible]al lead of course is 2gn, where g is the gage and [illegible] number of the frog. In practice these leads ar[illegible] down. Empirical rules for the standard gage ar[illegible] to find, but I have seen nothing that will apply [illegible] three-foot gage.

Yours truly, L. F. [illegible]

Mexico City, Mex., Nov. 24, 1905.

(Tratman's "Track" gives the following simple formulas for a short lead, or distance from the heel of the switch to the point of the frog:

$$\text{Short lead} = 6\,N + (L + F)$$

$$\text{Short lead} = 2\,N.\,(G. - \sqrt{G\,T}) + L.$$

In which N = number of frog. G = gage of track. L = length of switch rail. T = throw of switch. F = length of frog from point to [illegible]. —Ed.)

Formulas for Crown Thickness of Arches.

Sir: In your issue of Nov. 23 appeared a letter from Mr. Emile Low, in which he applies his formula for determining the crown thickness of masonry arches to the Cannington Viaduct. He also quotes my formula, but neglects to state that this formula is intended to apply to reinforced concrete arches only, and is particularly intended for highway bridges.

The writer does not believe it necessary to make the crown thickness of a reinforced arch so great as that of an un-reinforced arch, sufficient steel being used to take the tension if the center of pressure falls outside the middle third of the arch ring. He differs with Mr. Low also in regard to the effect of the rise of the arch on the crown thickness.

With solid spandrel filling, the negative bending moment at the crown usually increases with the rise, which offsets the effect of the lessened horizontal thrust. Hence the thickness of the arch ring may remain constant for the same span. Of course for extreme cases or unusual conditions the careful engineer will make independent investigations.

Respectfully yours, F. F. Weld.

Detroit, Mich., Dec. 1, 1905.

The Relative Activity of Portland and Natural Cement.

Sir: May I offer a word in supplement to your reply to "Young Engineer" who inquired in your last issue about the relative speed of setting of Portland and natural cements. He referred to an apparent discrepancy upon this point between the facts and the text-books, a discrepancy which, I am sure, has puzzled others besides him.

The confusion arises, in my opinion, not only from the causes which you cite, but also, in some cases, from a failure to observe that the laboratory men and text-book writers make a clear distinction between "setting" and "hardening." In their technical sense, a cement mortar is said to have "set hard when it will resist a slight pressure of the thumb nail" (Baker's "Masonry Construction," p. 58). Real hardening is then but just begun, and continues for months or years.

If setting be understood in the laboratory sense, then, no doubt, natural cements are, as you and the text-books say, the quicker setting of the two; but if setting be understood to mean an acquirement of real load-carrying capacity, then Portland cement should be spoken of as the quicker setting. In other words, while natural cement may show signs of hardening more quickly than Portland, Portland will very far outstrip it within 24 hours, never, so far as we know, to be overtaken.

Yours very truly,

L. J. Johnson.

Harvard University, Cambridge, Mass., Nov. 23, 1905.

Sir: In reply to "Young Engineer" in your issue of Nov. 16, will say that I have had more than 25 years' experience with cements, and find that in testing cements neat that the natural cements will commence to set, as a rule, somewhat quicker than the Portland cements, but that the Portland cements harden more rapidly after they begin to set than do the natural cements. In using cement in mortar in actual work, I find that the Portland cements set more quickly than the natural cements, and harden very much more rapidly after they begin to set, so that the mortar made with Portland cement will be as hard and strong in one week as the natural cement mortar will be in two or three months. I also find that most Portland cement mortar gets nearly its full strength in one month, while it takes the natural cement mortar from six months to a year or more, to get nearly its full strength; also, that many of the Portland cements weaken more or less, after a year or two, while the natural cements continue to slowly increase in strength, so that they get to be as strong, if not stronger, than the Portlands.

If great strength is needed in a mortar in a short

time, the Portlands are the better to use, but if the ultimate strength is not needed for a few months, the natural cements will do fully as well, and at less cost.

Yours very truly, Edwin Puzey.

141 Erie St., Buffalo, N. Y., Nov. 18, 1905.

Concrete Block Construction and the Architects.

Sir: On pages 553 and 554 of your issue of Nov. 23 is printed the best article by far of any that has ever appeared to my knowledge regarding the hollow concrete block. The writer of that article seems to grasp the situation and puts it properly before your readers, although very few, comparatively, will understand the fine points he makes. It is gratifying to know that the public is beginning to realize what possibilities remain for the industry, and that the profession, either architect or builder, begins to see what is necessary to make the industry what it will be and should be. Our twelve years of constant work in this one line has taught us what must be done in every instance, and every branch to avoid failure.

It would be hard to mention one sentence that deserves more commendation than another in the whole article, but the expression, "to compete with brickwork the manufacturer was compelled to sacrifice everything to cheapness," appeals to the founders of the industry quite forcibly from the fact that it was so hard at first to induce anybody to invest, or even to acknowledge that block construction was of any value; it was only after several buildings were completed by us that we could secure one cent as a loan. Gradually its merits dawned on those who wished to build and were short of money, and this cheapness being put before them by the manufacturers who were anxious to underbid brick finally took root and sprouted. "Everything sacrificed for cheapness," although perfectly conscious as to the ultimate success and gradually learning the fine points in the business which the author of the article seems to understand, and which not one in a hundred does understand, the industry was at last launched for a successful voyage. The recognition of the unit of measurement and its multiple was the first grand stride; this is not considered of much importance, even to-day by nine-tenths of the hollow block men and by but a very few of those who advertise to sell machines. Other features of just as much importance are not practiced, or, by many, even mentioned.

Of all the writers for the prize offered by Engineering News not one touched the vital points or seemed to understand the lines which must be followed to make even a partial success and much less to understand the general conception of the author as disclosed by this article, and he is entitled to both prizes. Kindly give me his name as I have not the slightest idea whom he may be; whoever he is he will fill a niche in the industry that is sadly vacant. It is very easy to compile an article from a lot of trade catalogues, but these are often very deceptive and to the practical man of little value.

There are those who can take care of the mechanical proposition, who know, and know they know, what lines must be followed for success; but other branches of endeavor calling for talent in disseminating the knowledge which is gained by experience, and imparted by the practical men, must follow, and in this branch of the work no article has appeared which equals this one without a name.

Most respectfully yours, H. S. Palmer.

Harmon S. Palmer Hollow Concrete Building Block Co., Washington, D. C., Nov. 23, 1905.

(The name of the author of the article referred to by our correspondent is: The Editors of Engineering News.—Ed.)

The Relative Merits of Contact Beds and Percolating Filters for Sewage Treatment.

Sir: I have read with much interest the editorial remarks in your issue of Oct. 19 on a paper I read before the Association of Sewage Works Managers in this country. I have also read some remarks by Dr. Gilbert Fowler, of Manchester, published in the same edition of your paper, with your editorial comments on them, and also on the general question of sewage disposal.

I am tempted by these to ask you to insert a few lines from me further elucidating what I believe to be the position of the sewage problem, at least in this country. When I said that "contact beds as a system for sewage resolution and purification are dead," I believe I voiced the opinion of the majority of those having large experience of modern sewage purification here. I know it is the opinion of Dr. George Reid, the County Medical Officer of Health for Staffordshire, who has had a very wide experience of sewage disposal in that large and populous county, while the immense works now being instituted by Mr. Watson near Birmingham, where the sewage of over three quarters of a million is dealt with, and where nothing but percolating filters are being put down after prolonged experiments, seems to point in the same direction.

My reason for saying that contact beds were dead as a method for sewage disposal is that having had experience, not only of all the sewage works in the County of Warwick, but in many other important cities and towns of this country, I find that better results are invariably obtained with percolating filters than with contact beds. Let me give the latest illustration of this that has come to my knowledge: A few years ago, just before the advent of the percolating filter, the town of Nuneaton put down a contact filter bed plant. This, owing to its natural position, is one of the best examples of this kind of purification to be found. Recently, an experimental percolating filter has been erected, so that the two methods of sewage purification, dealing with the same sewage, under the same conditions, are going on side by side, and the results in the case of the percolating filter are infinitely superior to those of the double contact beds.

Just a word or two as to the colloidal matter referred to by Dr. Fowler. My experience is that such precipitation of colloidal matter does not take place when good effluents are produced, as they can easily be produced by *percolating filters;* but it must be remembered that many so-called percolating beds are, in my opinion, improperly constructed. The materials are unsuitable in quality, and too large in size, and I believe, as I endeavored to show in my paper, that the main feature in a percolating bed is the small size and hardness of the particles. I was one of the first to bring to the public notice the great value of coal as a filtering medium, and I believe that to a very large extent the power that this substance has of producing a really high class effluent is due to the fact that when coal is used for this purpose it is always in a finely divided state. There can be no doubt, too, that in the early days of artificial filtration of sewage, polarite filters were, when they were not overworked, very efficacious, and I believe the high quality of their work was due also to this reason.

That there may be advantages under certain conditions in contact beds I will not deny, but the cases in which this is so must be very few indeed, and by the improvements now in vogue in distributing the sewage on the surface of the filter I can hardly conceive of any condition when, as the statutory officer for the protection of rivers in Warwickshire, I should without protest permit contact beds to be set up in place of percolating filters.

It is a matter of regret that I have never yet been able to see the methods of sewage treatment in vogue in the United States, but I hope that at some future date I may have the opportunity of doing so.

I remain, your obedient servant,

A. Bostock Hill,

Professor of Hygiene and Public Health, University of Birmingham.

14 Temple St., Birmingham, Eng., Nov. 10, 1905.

(We are pleased to give space to Professor Hill's further elucidation of his ideas on the relative position of contact beds and contact filters in England. On a number of previous occasions we have stated that of late percolating filters seemed to be more in favor than contact beds, but we think that some of the evidence as to the comparative merits of the two is still lacking, particularly in the matter of combined capital and operating charges per unit of sewage of the same character treated to the same degree of purity. Either method, when combined with the necessary preliminary or after treatment, seems to be capable of producing such results as local conditions may require, provided sufficiently large areas per 1,000,000 gallons of sewage are provided; but at what relative outlay of capital and labor, including not only the beds or filters and their various accessories?

It is quite true that Birmingham is building percolating filters; and it is equally notable that Manchester has built contact beds, and that Sheffield has within a few months obtained Local Government Board sanction for contact beds, and that, too, with only sedimentation for preliminary treatment. It remains to be seen, after completion and a few years of operation, what the relative efficiency and cost at these several works will be, in so far as local conditions will permit comparisons.

A reduction in size of grain and an increase in the hardness of the medium of percolating filters, as now generally constituted abroad, both seem desirable. Carry such changes far enough and there would result the American type of intermittent sand filter, except that it would be provided with sprinkling apparatus above and freer ventilation and drainage below than is the case with intermittent filters.

In conclusion, we trust that nothing we have said will lessen the weight which should properly be given to the extended observations and experience of Professor Hill and Dr. Reid, nor to the example set by Mr. Watson at Birmingham, all in favor of percolating filters.—Ed.)

Labor and Contract Conditions in the Philippines.

Sir: The articles, "Labor Conditions in the Philippines" and "Sanitary Conditions as Affecting Contracts for Work in the Philippine Islands," in the Engineering News for Nov. 23, contain, I believe, the most authentic information, on the subjects treated, that has been published by a private press.

The large amount of material, bearing directly and indirectly upon these subjects, which has been published can be divided into two main classes: (1) Government Reports and (2) current periodicals, both daily papers and magazines. As is usual the former has had a very small, the latter a very large circulation. Moreover, the general trend of the latter has been to minimize the possibilities in and probabilities of the native laborer.

In the article on Sanitation Major Carter writes: "The statements made by volunteers and others in the period beginning 1898 and ending in 1902 or 1903, were in the main correct and just;" he then proceeds to give in general the conditions under which such statements were made.

In attempting to do no injustice to "volunteers and others" (the writer belonged to both classes during the years 1898 to 1901) Major Carter has not said that many of these reports were based on short experience, many made by men who had never before been in charge of labor and who were gaining their first experience in a position which would have been difficult for an experienced man. The wonder is that these reports were not more adverse.

Sections of the earlier Government Reports are open to this same criticism.

An experience of two years in handling native Filipino labor firmly convinced the writer that, dollar for dollar paid in wages, the native was the best labor available at that time. All the evidence in Mr. Beardsely's article substantiates that conviction and goes far to prove that such is the condition with a constantly improving labor market. The labor obtainable during the first years of our occupancy was, from the nature of the existing conditions, of an inferior grade, but even under those conditions men remained in the Government employ, in the writer's department, during the entire time before mentioned, or from December, 1898, to January, 1901.

After the middle of the year 1899 little trouble was experienced with my force, numbering from 80 to 130 men, until the fall of 1900. Trouble arose then because an American, who was put in charge of about thirty natives doing street work, was unable to speak enough Spanish to make them understand his orders, and, instead of showing by example what he wanted, he tried to cuff them into obedience. This action on the part of the foreman caused a strike; the entire force on his work coming to my house and in a most orderly manner presenting, through a spokesman, their side of the trouble. Upon being assured that the matter would be personally investigated they returned to work and, the investigation being made, the same men were laboring there when I left for the United States. From personal experience ludicrous cases of the misuse of tools, such as mattock and pick, could be given to illustrate one clause of Mr. Beardsley's article.

Mr. D. M. Carmen has been in the contracting business in the Philippines since 1898 or 1899 and Mr. H. L. Higgins was holding his present position at the time of our first occupation of Manila. Both of these men (quoted in Mr. Beardsley's article.—Ed.) are in a position to speak knowingly on the labor question.

The Philippines will become, under intelligent management, the most productive and beautiful archipelago in the tropics. The Government Reports show that Manila was not an unhealthful place in 1899-1900; it certainly will not grow worse with improved sanitary conditions and increased knowledge among the natives.

Having been on the minority side for so long it gives me much pleasure to read the articles referred to and be able to congratulate you upon securing them.

I am, yours very respectfully,

T. M. Ripley, Assoc. M. Am. Soc. C. E.

52 Chestnut St., Albany, N. Y., Nov. 27, 1905.

Notes and Queries.

The volume of water in Marston Lake treated with copper sulphate by the Denver Union Water Co., as described in our issue of Nov. 29, was 4,500,000,000 gallons.

H. T. T. Tacoma, Wash., inquires why engineering work in America is not paid for on a percentage basis, as it usually is in Great Britain.

In reply, it may be said that a considerable amount of engineering work is now done on a percentage basis and there is perhaps a tendency toward greater use of this basis in fixing fees. It is difficult for engineers of high standing to bring their clients to the percentage basis,

however, when other reputable engineers are willing to bid to do the work for a lump sum.

In our issue of Nov. 23, in printing our report of the address of Hon. Wm. H. Taft on the progress made at Panama, a typographical error occurred on p. 551, near the end of the address. The paragraph reading: "I visited the Isthmus a year ago, and again this year. I am unable to see a marked difference, etc.," should read, of course, "I am able." The context made the Secretary's meaning clear enough, we should suppose; but the error is claimed to have misled a prominent daily newspaper, which made the statement a text for some unmerited strictures on the canal administration. We therefore give prominence to the correction.

In a paper on "Notes on Workmanship in Riveting," by Mr. A. J. Himes, M. Am. Soc. C. E., published in our issue of Oct. 19, allusion was made to work on bridges over the Erie Canal, where engineering methods were formerly very lax. In fairness to those now in charge of that work it should be put on record that in 1899, under the administration of Mr. Edward A. Bond as State Engineer, the legislature passed an act providing for the employment of bridge designers to have special charge of all bridge work under supervision of the State Engineer. In August of that year Mr. Wm. R. Davis, Assoc. M. Am. Soc. C. E., was placed in charge of this work and since that time all material and workmanship on state bridges has been subject to systematic inspection.

R. R., Cumberland, Md., writes concerning the shrinkage of earthwork:

It seems hard to conceive that earth as naturally deposited should not be as dense, or rather more dense, than when artificially deposited, as the natural agencies have had so much longer time to exert their influence.

The compression that is due to the traffic during construction would be the only circumstance to favor a shrinking from cut to fill; but as there always proves to be shrinking after the work has been completed, it may be presumed that, notwithstanding all artificial means, the paramount causes to the shrinkage of an earthwork are those that have been working for a so much longer time.

The probable reason for the unquestioned shrinkage of almost all varieties of earth when excavated from natural bed and compacted in a fill, is that earth as placed by nature is quite porous, due probably to the action of percolating water during long periods of time. When earth is excavated these porous channels are broken up and the mass when compacted has a smaller percentage of voids than in its original condition.

THE AUSTERLITZ BRIDGE ACROSS THE SEINE FOR THE METROPOLITAN RAILWAY OF PARIS.

By Réné Bonnin.*

Line No. 5 of the Metropolitan railway of Paris, which connects the Northern railway station with the Place d'Italie, by way of the Magenta, Richard Lenoir and L'Hopital boulevards, crosses the Seine, on a viaduct, about 200 meters above the Austerlitz Bridge. This viaduct, the tunnel crossing of the Seine, now under construction, between the Chatelet and St. Michael stations, of line No. 3, and the viaduct crossing of line No. 2, at Passy, are the most important works of the Paris Metropolitan Railway. The first of these works offers certain technically new and interesting features.

The problem to be solved was to cross the Seine by means of a metallic span of 140 m. between abutments, with no point of support in the river and with the under side of the deck at not more than 12 m. above the level of standard low water, so as to avoid too important approach bridges and especially too steep slopes.

It was quite impossible, under these unusual conditions, to build a bridge with arches underneath the deck. It was absolutely necessary to resort to arches above the deck and to support the latter by suspension rods fastened to the arches. It is a type of bridge similar to the one adopted in 1900, by the Western Ry. Co. for crossing the Seine at Passy, on the line leading from Courcelles to the Champ de Mars, and to those built in recent years across the Rhine by German engineers.

Several arrangements might have been adopted for these elevated arches. They might be let in on the abutments to the springing lines. But as the calculation for these arches could not be made by the rules of statics, resort must needs be had to the laws and formulas of elasticity; moreover, any settlement at the points of support on the abutments makes a sensible change in the results of calculation and work of the arch; finally changes of temperature have an important influence on this same work of the arch and make necessary a great increase in its section. The consequent result is a heavy arch with, often, a form which is ungainly from the aesthetic point of view.

The arch could be built with hinges at each abutment. This type was adopted by the Western Ry. Co. for the Passy bridge, and by German engineers for the bridges over the Rhine at Bonn, where the span is 187.20 m., and at Dusseldorf, where it is 181.30 m. But in this case, too, the arch cannot be calculated without resorting to the formulae of elasticity and the variations of temperature have, as in the preceding case, a marked influence on the section to be given to the arch. Its weight is still large, although less than that obtained when the arches are imbedded.

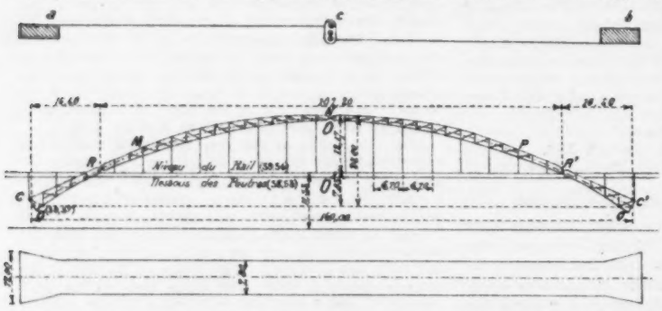


FIG. 2. CONTRACTORS' SUBSTITUTED PLAN FOR AUSTERLITZ BRIDGE.

Finally the arches may have pivots at the springing lines and at the crown. They can then be calculated by the laws of statics without resorting to the laws of elasticity and, moreover, changes of temperature have no influence on the stresses in the individual members of the arch. The engineers of the Metropolitan concluded, in their preliminary project, to adopt this type of bridge, little used as yet in Europe when it is a question of a railway bridge. As shown in Fig. 1 this bridge was composed of arches having articulations at the springing lines and at the crown. Each arch was formed of two crescent-shaped lattice girders. The span at the articulations of the springing lines was 140 m., and the rise was 28 m., or a ratio of $\frac{1}{5}$. The deck of the bridge, of which the lowest point was 11.33 m. above standard low water, was hung from the arches by suspension rods.

Various contractors were invited to enter into a competition based on the above preliminary plan. The award was made to the Levallois-Perret Construction Society, which proposed a happy modification of the project, this modification was accepted by the Metropolitan's engineers. The modification is this:

It is a well-known fact that the pressure line changes its position with the position of the live load, in this case the train, on the bridge, and that the difference between the ordinates of this line and those of the neutral axis of the arch will be greater, for a given load, as the span (between hinges) is greater. On the other hand, as this span must, in no case, go outside of the arch,* it follows that, the greater distance between the articulations at the springing lines, the greater must be the height of the crescent-shaped trusses which form the arch; the greater also must be the weight of the arch. The best solution consists, then, in reducing the span between the springing line articulations and the Levallois-Perret Société founded its project on this fact, submitting to the directors of the Metropolitan the project shown by Fig. 2, a project which the directors adopted and which is now in process of being carried out.

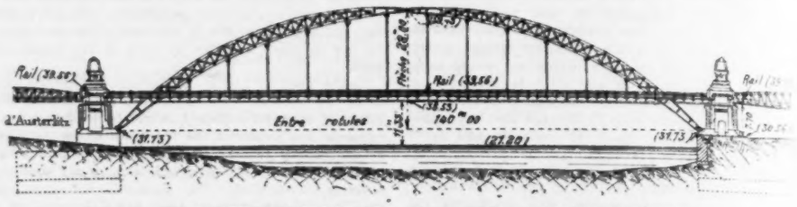


FIG. 1. ENGINEERS' PRELIMINARY PLAN FOR AUSTERLITZ BRIDGE.

As seen by Fig. 2, the span of the arch with three hinges is reduced to 107.2 m., and the thrust of this arch is thrown, at the hinges of the springing lines, on brackets which reach, for a length of 16.4 m., to the abutment on which they bear by means of two skewbacks fitted with adjusting keys. These skewbacks are of large area so as to distribute the pressure over the masonry as evenly as possible. In this way the height of the crescent-shaped trusses, which form each semi-arch, could be reduced and with it the weight, thus giving the whole structure the appearance of very decided lightness. The rise of the three-hinged arch is 11.726 m., or a ratio of $\frac{11.726}{107.20} = \frac{1}{9.142}$ for a span of 107.2 m. But this way of building elevated arches, with or without supporting brackets, and with a deck suspended by means of suspension bars, involves certain complications in construction which require the adoption of a few special arrangements. Among these are two of chief importance—one relating to wind pressures, the other relating to expansions and contractions arising from changes of temperature.

When a bridge is built with an arch carrying the floor, or deck, above it, the wind pressures acting on the arch and the deck are distributed over the points of support on the abutments, by means of a continuous bracing which connects the different arches forming the framework of the bridge throughout their entire length. When the arches rise above the deck, this continuity of bracing cannot be kept up throughout the entire length, as a free space, necessary for the passage of trains, must be left. Thus, at the Austerlitz Bridge this bracing could only be put in over the length MNP (Fig. 2), the two open spaces RM and PR', having to be left in order to let trains go by. As the result of this interruption of con-

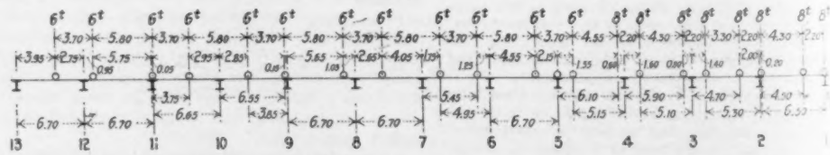


FIG. 3. TRAIN LOAD DIAGRAM USED IN CALCULATING AUSTERLITZ BRIDGE.

*61 Rue Louis Blanc, Paris, France.

*That is, if it be desired to avoid tension in the chords. —Ed.

tinuity in the bracing of the arches, the wind pressure on this part MNP of the arch could not be carried directly to the abutments, so the deck of the bridge itself had to be used for the purpose. Each of the frames, formed by the vertical suspension rods, the deck girders and the top ties, which hold this part of the arches, had to be made stiff enough to transmit to the deck the efforts of the wind against the arches. In fact, it is the deck which, by its floor of sheet steel plates, to which the rails are fastened, and its bracing underneath, carries to the abutments all the wind pressure acting on the arch and on all the other parts of the framework.

A second special arrangement had to be adopted in order to allow for changes of temperature. The deck increases or diminishes in length under the influence of changes of temperature, and produces on the vertical posts, which connect it with the arch, bending strains, which are the greater as the distance between the primitive position of the upright and that resulting from this change of temperature is greater. Hence this distance should be made as small as may be. To fasten permanently the deck to the arches at the points where they intersect cannot be thought of, because, in this case, the deck forming the tie rod of the arch would give in this arch great bending strains arising from changes of shape in the arch due either to the passage of loads or to changes of temperature. Nor can it be fastened to the arch at one of the points of intersection R or R', leaving the other point of intersection free; because, then, under the influence of changes in shape of the arch arising from an unsymmetrical live load and by the elongation of the deck, the vertical posts are subjected to great bending strains which reach their maximum at the last post next to the free articulation M or P.

In order to avoid these great bending strains, which would make it necessary to give the vertical posts a section out of reason, a happy arrangement, already used by the Levallois-Perret Society for the Passy viaduct across the Seine, was adopted. This arrangement consists in maintaining permanently the middle of the deck to the middle of the span OO^1. The change in the positions of the lower ends of the vertical posts under the influence of the elongation of the deck is thus reduced to the elongation of the half chord of the arch, instead of the whole chord, and the bending strains are markedly reduced. To keep the middle of the deck permanently at the center of the span OO^1, two tie rods are fastened at two points a and b (Fig. 2) of the skewbacks of the arches, the other ends being made fast to a balance lever of which the axis c is made fast to the deck, at the middle of the span.

The deck should fulfill a last condition. The hinge at the top of the arch rises or falls under the influence of changes of temperature. When, as a result of a rise in temperature the hinge rises, the deck which is connected to the arch by the vertical posts follows this ascending movement and has at its middle a great bending stress which must be taken into consideration. The deck might have been cut in the middle by a hinge, but it was preferred to give to the vertical moment of inertia of the outside beams of the deck such a value that the extra work of the beam, under this bending stress, should remain within suitable limits. Moreover, the deck stringers of the middle panel, that is the one which receives the maximum bending strain due to the rising of the top hinge are jointed on the deck girders so as to facilitate the vertical movement of the deck.

DESCRIPTION OF THE ARCHES.

In calculating the sections to be given to the arches, the different strains coming on these arches were considered. These strains are due:

(1) To the weight of the metallic part of the bridge which, per running meter, may be distributed as follows: Metal, 4,400 kgs. per meter; track, rail supplying current, side passages, 550 kgs. per meter, or a total of 4,950 kgs. per meter, or, say, 5,000 kgs., giving 2,500 kgs. per running meter of arch, these being two in number.

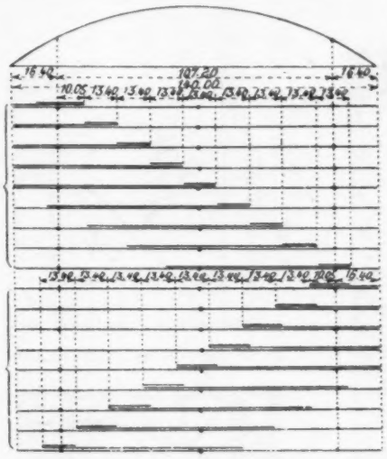


Fig. 4. Train Distribution Diagram Used in Calculating Austerlitz Bridge.

(2) To the live load, which is composed of two typical trains of the Metropolitan railway, the scheme of which, Fig. 3, shows the composition, as well as the distribution of the load per axle. It has been assumed, for purposes of calculation, that two trains were running in opposite directions on the bridge, and occupying nine different positions, as shown in Fig. 4. It has been assumed, further, that the effects to which each train gave rise in an arch are proportional to their respective distances from these arches, so that the part of load P referred to the nearer arch is, Fig. 5,

$$R = P\frac{5.35}{7.80} = 0.686\ P.$$

and for the more distant arch

$$R' = P\frac{2.45}{7.80} = 0.314\ P.$$

(3) To the strains due to wind pressure on the different parts of the arch, strains of which the result is to overload the leeward arch and to relieve the windward arch. Two suppositions have been made: (a) A wind causing a pressure of 270 kgs. per square meter of surface, with no train running on the bridge; (b) a wind giving a pressure of 170 kgs. per square meter, with one train running on the bridge.

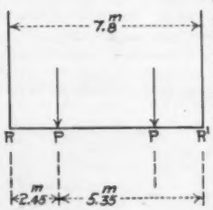


Fig. 6.

Combining these different strains, their maximum could be determined for each of the sections of the arch and, consequently, the cross section to be given to the chords to keep them within the working limits required by the regulations of 1891. This maximum work is, for the upper chord, 9.3 kgs. per sq. mm. (13,226 lbs. per sq. in.), including the wind, and 8.3 kgs. (11,805 lbs.) exclusive of the wind. These quantities become for the lower chord, respectively, 9.45 and 8.1 kgs. (13,441 and 11,521 lbs.) The maximum work of the lattice bars is 6.7 kgs. (9,530 lbs.) with wind pressure and 5.2 kgs. (7,396 lbs.) without.

The two arches are 7.8 m. apart between centers. Their neutral axis is a parabola of 140 m. span and 20 m. rise, the ratio being $\frac{140}{20} = 7$.

These arches, as has been said already, are composed of a central part, with three hinges, of 107.20 m. span and 11.726 m. rise, supported at the springing lines by brackets which bear upon the abutments. The two half trusses of the three-hinged arch, Fig. 6, are crescent-shaped and have their maximum height at the third vertical post. They have latticed webs, except near joints, where the lattice work is replaced by solid plates.

The outside and inside chords are shown in Fig. 7. Their great width makes them very stiff crossways; the posts and oblique bars are made of angle irons. Fig. 8 shows the hinge at the crown. The stresses which come on this point are:

(1) The dead load	306,250 kgs.
(2) The live load on the nearer track (6th case of the load)	130,000 "
(3) The live load on the further track (7th case of live load)	57,800 "
(4) The wind at 170 kgs., with live load	91,000 "
Total	585,050 kgs.

As the center hinge pin is 150 mm. in diameter and its length is 1,000 mm., the area of the longitudinal section is 150,000 sq. mm., hence the corresponding pressure per sq. mm. is 3.9 kgs. (5,547 lbs. per sq. in.).

The hinges at the springing lines are similarly arranged. The total stress brought on each is 633,400 kgs. As the length and diameter are the same as in the center hinge, the pressure per square millimeter is 4.2 kgs. (5,974 lbs.).

The brackets, which form skewbacks, and on which the hinges at the springing lines of the arch bear, are also of the box-girder form (Fig. 7). The height, which increases from the joint to where it bears on the abutment, is at this latter point 3.80 m. The brackets are latticed, like the trusses of the central part of the arch, except near the two ends where the lattice work is replaced by solid webs (Fig. 6). The maximum working stress of the upper chord of these brackets is 9.4 kgs. (13,370 lbs.) with wind and 7.4 kgs. (10,525 lbs.) without wind. In the lower chord these quantities are, respectively, 9.2 kgs. (13,085 lbs.) and 8.35 kgs. (11,876 lbs.)

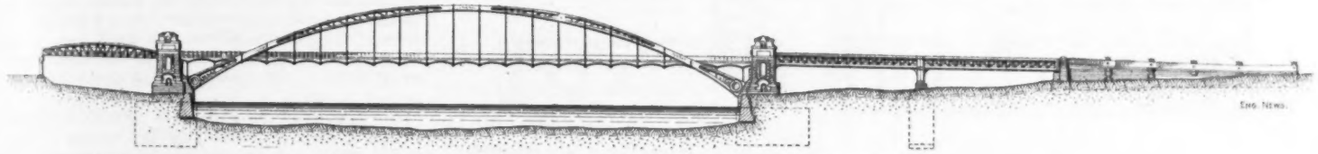


FIG. 5. GENERAL ELEVATION OF AUSTERLITZ BRIDGE.

Fig. 9 shows the arrangements of the skewbacks on the abutments. The pressure on the bearing at the extrados is 585,800 kgs. and the bearing surface on the masonry is 1.35 m. + 1.10 m.=14,850 cm., hence the pressure is 39.5 kgs. per sq. cm. (561.82 lbs. per sq. in.) In order to make sure of an even distribution of the pressure over the masonry, the space between the metal plate of the bracket and the stone of the abutment was filled with grout. The bearing key being 80 mm. wide by 1,000 mm. long, its total surface is 80,000 sq. m., hence the pressure per sq. mm. is 7.3 kgs. (10,383 lbs. per sq. in.) The pressure on the bearing at the introdos is only 453,800 kgs., so the unit pressures on the masonry and on the key are less than those at the extradosal bearing.

VERTICAL SUSPENSION POSTS.

The vertical posts which, fastened to the arch, support the deck, have to bear different stresses due to:

(1) The tension resulting from the weight of the deck which these posts must transmit to the arch, this weight, including that of the trains, being, for each post, 27,504 kgs.

(2) The wind pressure on the arches, which the posts have to transmit integrally to the deck, which alone must carry it to the abutments. Heavy transverse bending strains are the result

of this, these strains reaching a maximum for a wind pressure of 270 kgs. per sq. m. (55.30 lbs. per sq. ft.) In this case the supplementary work per square millimeter reaches 3.8 kgs. (=5,405 lbs. per sq. in.) at post 10.

(3) The displacement of the foot of the post, under the influence of changes of temperatnre of ± 30° C, with regard to the normal temperature, and of the displacement of the arch caused by an unsymmetrical live load. Under the influence of this displacement, the post takes the form shown by Fig. 10, in which λ represents the total displacement due to the expansion of the deck and to the displacement of the arch under the unsymmetrical load. Under these circumstances, there are produced at the two points of attachment strains represented by

$$P = \frac{12\,EI\lambda}{h^3},$$

in which E represents the coefficient of elasticity of steel, I the moment of inertia of the post and h the height of the post. This stress P produces a bending moment of,

$$M = P\frac{h}{2}$$

and the working stress per square millimeter in the post given by:

$$R = \frac{Mo}{I}.$$

As is seen, the stress P which causes the bending moment increases with the moment of inertia and with the displacement of the foot of the post; it is also inversely proportional to its height. Hence it is necessary, in order to reduce the supplementary stress due to this displacement, to diminish the displacement of the foot of the post, which is effected by fastening the deck at the middle of its length, and to reduce the moment of inertia of this post, by making its width, parallel to the axis of the bridge, as small as possible. By combining these different stresses it is found that post 6 undergoes a total stress of 9.4 kgs. per sq. m. (13,370 lbs. per sq. in.), including the wind pressure, and of this amount 3.7 kgs. (5,263 lbs.) are due to the displacement of the foot of the post under the influence of expansion.

In spite of the small width given to the posts, in order to reduce their moment of inertia, the strains resulting from the longitudinal displacement of the deck, from post 6 to the hinge at the springing lines, become too great, and posts 3, 4 and 5 are hinged at their points of attachment to the deck and to the arch. The maximum strain in these last posts reaches 9.2 kgs. (13,085 lbs.) at No. 5, wind pressure included. Fig. 11 shows the arrangement of post 6, the arrangement being exactly the same at the other fixed posts. These posts, 1 in. wide, are composed of two chords each made of two angles of 70 × 70 × 8 mm. connected by lattice bars riveted to a web which, in its turn, is riveted between the angles of the chords. Fig. 12 shows the hinged post No. 3.

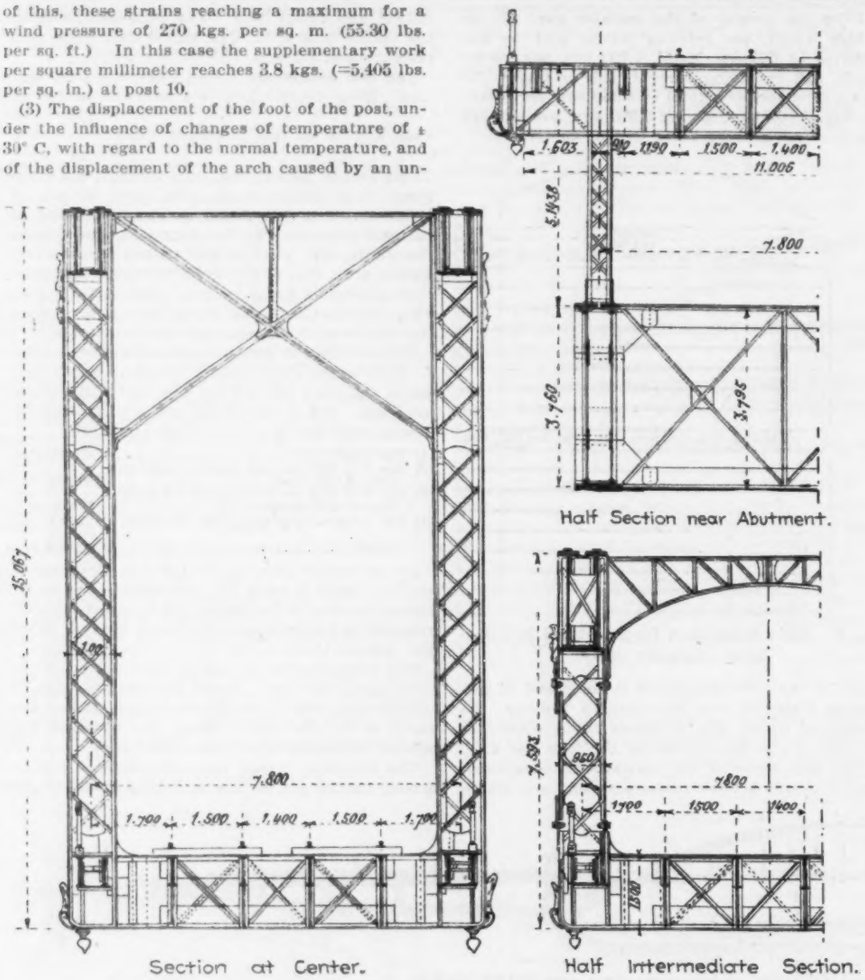


FIG. 7. TYPICAL CROSS-SECTIONS OF ARCH SPAN, AUSTERLITZ BRIDGE.

DECK.

The deck which carries the rails of the track serves also to take to the abutments all the pressures due to the wind acting on the frame of the bridge. It must have, therefore, a transverse strength sufficient to stand the bending and shearing moments due to the wind.

For this purpose it has received the form of a beam laid on its side and having the two extreme chords formed of an outside beam, of the shape of a box girder, 0.60 m. high (Fig. 7). This outside beam passes through the feet of the posts which connect the deck with the arch, and is fastened to the deck. The outside beams are connected together by deck girders 1.50 m. high, fastened to each post and, like these, set at a distance apart of 6.70 m. between centers. Stringers 0.60 m. high, like the outside beams, connect these deck girders longitudinally and are placed under each line of rails. A floor of sheet-steel plates on top of, and a system of diagonal braces and counterbraces underneath the stringers and outside beams each takes one half of the shearing moments due to the wind. The bending moments are borne by the outside beams.

The outside beams are, as shown by Fig. 13, fastened to both the extrados and intrados of the arches and extend to the masonry abutments to which they are anchored. But, in order to prevent the outside beams from forming tie rods for the arches and causing supplementary strains in the latter, they are cut at A, near the intrados m n of the arch. They can thus slide longitudinally in the fixed parts A m n A', under the influence of changes of temperature which lengthen or shorten the deck from its middle which is fastened to the expansion tie rods a b placed inside of the outside beams, the fixed ends a of these tie rods being attached to the immovable parts of the outside beams riveted to the intrados m n of the arch.

The expansion tie rods have also to bear the strains arising from applying the brakes on the trains which are running on the central part of the deck and which impart a longitudinal motion to the deck. The weight of a train being 136 tons and the maximum coefficient of adhesion which can be assumed has 0.25, the longitudinal strain caused by the application of the brakes will be $\frac{136{,}000}{4} = 34{,}000$ kgs., a strain which is transmitted to the four points at which the expansion

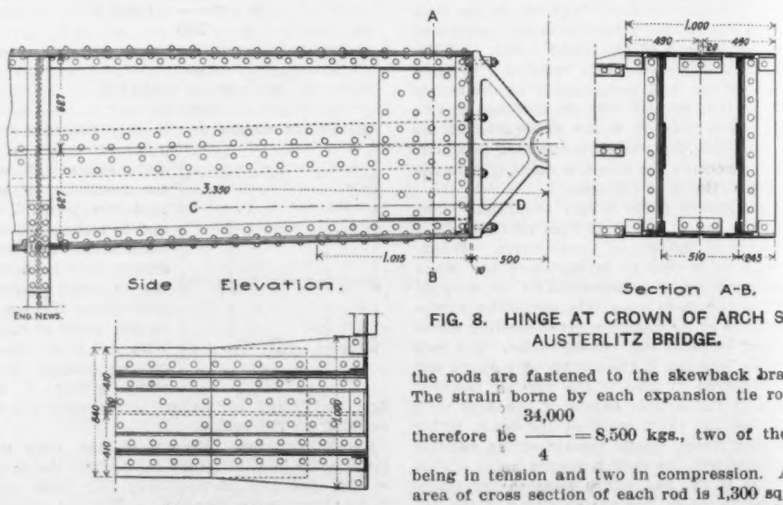


FIG. 8. HINGE AT CROWN OF ARCH SPAN, AUSTERLITZ BRIDGE.

the rods are fastened to the skewback brackets. The strain borne by each expansion tie rod will therefore be $\frac{34{,}000}{4} = 8{,}500$ kgs., two of the rods being in tension and two in compression. As the area of cross section of each rod is 1,300 sq. mm., the resulting work per square millimeter is 6.5 kgs. (9,245 lbs. per sq. in.) In order to avoid the bending moment due to compression, the rods are guided throughout their entire length. As to the central axis of the balance lever, fastened to the middle of the outside beam, it bears a strain of

$\frac{34,000}{2}$ = 17,000 kgs. Its area of cross section being 7,853 sq. mm., the shearing stress is 2.2 kgs. (3,129 lbs.)

The part of the deck fastened to the central movable part of the outside beams and which ends

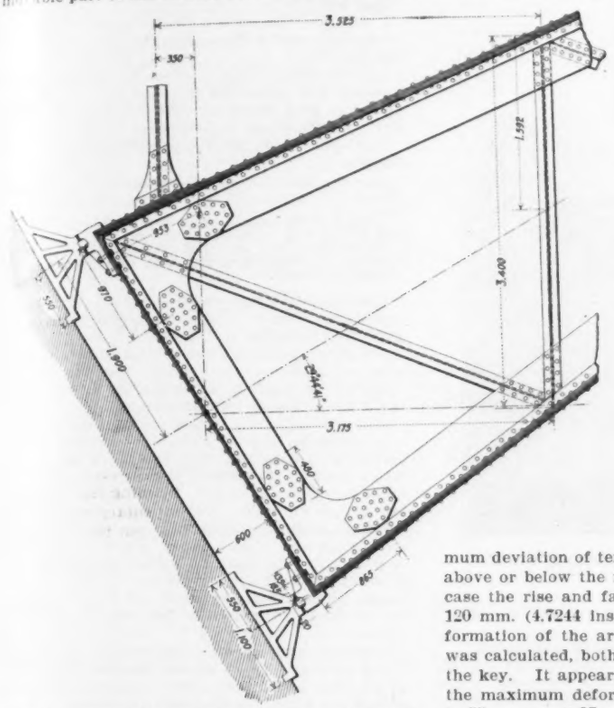


Fig. 9. Skewback Bracket Bearing on Abutment, Austerlitz Bridge.

at the cross beam f, is connected at o to the fixed deck girder e riveted to the arch, by means of a dowel which penetrates into a bracket plate let into the deck girder e. This dowel is taken from among the plates which allow an extension longitudinally of the central part of the deck, but prevent any motion laterally. The bracing of the deck of the central arch is entirely independent of that of the deck of the skewback brackets.

This dowel, which is the connecting link between the two sets of braces and which, by its lateral pressure on the bracket plate under the influence of the wind thrust on the central part of the deck, carries the resulting pressures to the fixed part H H′ K K′ of this same deck and, by means of the bracing, to the outside beam H K, anchored at K in the masonry of the abutment.

The maximum bending moment of the outside beam of the central arch, due to the wind pressure, occurs at the middle of the span. It amounts to 1,720,000 kilogrameters. As the cross section of the outside beam at this point is 25,184 sq. mm., the stress per sq. mm. is 8.8 kgs. (12,516 lbs. per sq. in.) But to this stress should be added that of which mention was made at the beginning and which results from the bending moment arising from the lifting of the key joint which,

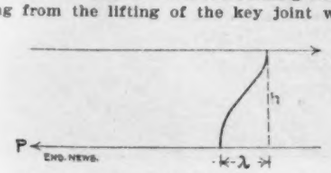


Fig. 10.

under the influence of an increase of temperature, carries in its movement the floor and the outside beams. This additional work is of 2 kgs. per sq. mm. (2,845 lbs. per sq. in.), which brings the maximum stress of the outside beam up to 10.8 kgs. (15,361 lbs.), a maximum which will probably never be reached, because it is very unlikely that a stress of 8.8 kgs. (12,516 lbs.) due to a maximum wind pressure of 270 kgs. per sq. m. (55.3 lbs. per sq. ft.) will ever happen with extreme variations of temperature.

It should be added that, although the outside beams riveted to the extrados of the arch and anchored in the masonry have not been considered in the calculation of the sections of the brackets of the arch, whose stability is assured without counting on them, this connection of the outside beams with the arch is, in fact, an additional element of safety and, especially, of stiffness.

DEFORMATIONS.

The deformation due to the dead load was first calculated and, for that purpose, the shortening of the chord of the arch under the influence of the strains of compression caused in the arch by the permanent load was calculated. This shortening is 22.84 mm. It was sufficient, to overcome it, to reduce by this amount, during construction, the distance between the supports of the abutments. Next the deformation of the arch due to a maximum deviation of temperature of 30° C (=54° F.) above or below the mean was calculated. In this case the rise and fall of the hinge at the key is 120 mm. (4.7244 ins.) Finally, the maximum deformation of the arch arising from the live load was calculated, both at the springing lines and at the key. It appears from these calculations that the maximum deformation at the springing lines is 55 mm., and 37 mm. at the key.

ERECTION OF THE BRIDGE.

The bridge was erected by means of a service bridge (Fig. 14) across the entire width of the river, and having four openings each of 18 m. span for navigation. The metallic parts were landed at the Austerlitz quay, whence they were lifted by derricks to small cars which, running on the service bridge, carried them to where they were to be used. Work was begun by erecting the supports of the arches and of the deck, the different parts of which were set by means of a traveler, shown in Fig. 15.

The brackets of the arches being up and the deck with its metallic floor being completely riveted, the latter was used as a platform for raising the arches, an operation which was performed by means of a derrick on a tower, composed of four uprights, and sufficiently high for setting the topmost pieces, those at the crown. As the work on the arch advanced the sections were made to rest on the vertical suspension posts, care being taken to steady the latter by means of wooden posts and St. Andrews' crosses.

ABUTMENTS.

Fig. 16 represents the abutments serving as points of supports for the skewbacks of the arches. These abutments rest on a mass of ma-

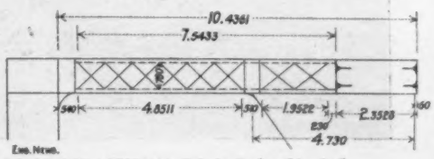


Fig. 11. Fixed Suspender, Austerlitz Bridge.

sonry on a rectangular steel caisson founded by means of compressed air. The caissons had rounded corners, 18 m. long and 22 m. wide, giving a surface of 396 sq. m. The caissons rest on a bed of hard limestone encountered, on the right bank at 9.70 m., and on the left bank at 11.05 m. below the low water surface of the Seine. The caisson was sunk evenly and without any special difficulty, at the rate of about 25 cm. (=10 ins.) a day.

COST.

The total cost may be estimated at 1,450,000 francs, of which the abutments cost 1,000,000 and the metallic parts 450,000 francs.

ENGINEERING CONTRACTS.*

By J. A. L. Waddell,† M. Am. Soc. C. E.

The dividing line between specifications and contracts is most difficult to draw, for in any particular case two engineers will rarely agree as to what clauses pertain properly to the specifications and what to the contract of which the specifications form a part. Some engineers prefer to throw nearly everything into the specifications and thus keep the size of the contract proper as small as possible, while others make the latter very extensive by including in it many clauses that are ordinarily found in the specifications. Again, others make a practice of repeating in the contract certain clauses that have already been covered in the specifications. In my opinion, the last mentioned method is open to criticism in that it is liable to result in conflicting clauses.

Before proceeding to the direct treatment of my subject I shall endeavor to make clear to you the method

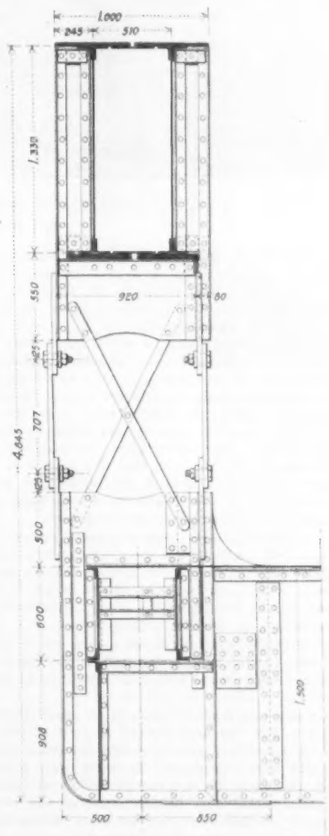


Fig. 12. Hinged Suspender, Austerlitz Bridge.

that I have adopted for locating the dividing line between specifications and contracts. I say "endeavor" advisedly, for I am not sure that I can always give a satisfactory rule or reason for any particular division; because absolute consistency is an attribute that, strive as one will to attain it, lies ever just beyond reach.

My preference is to throw as much of the matter as possible into the specifications and reduce the size of the contract proper to a minimum, avoiding repetition of statement in the two parts of the work, but of necessity treating certain subjects in both parts, though from different points of view. There is no doubt about the proper place for most of the topics or headings, but in certain cases there are plausible reasons for locating them in either division. All clauses that relate to methods of construction, qualities of materials, character and excellence of the work, rules limiting the functions and powers of the Contractor and defining the authority of the Engineer, directions to bidders, and transportation of men and materials unquestionably belong to the

*Condensed from a lecture delivered at the Rensselaer Polytechnic Institute.

†Consulting Engineer, Kansas City, Mo.

specifications; but such clauses as those relative to adherence to specifications, alteration of plans, damages, extras, payments, responsibility for accidents, the spirt of the specifications, strictness of inspection, liquidated damages, scope of the contract, and time of completion might perhaps be properly inserted in either division. My custom, however, is to include all of these clauses and others of like character and scope in the specifications.

Nine out of ten of the contracts that an engineer has to prepare are in connection with construction, and an intelligent specialist soon learns how to prepare satisfactory specifications and contracts for all ordinary kinds of work; but this style of contract is by no means the only type with which an engineer is concerned, for he is sometimes called upon to draft agreements between promoters of enterprises and capitalists, between himself and promoters of enterprises, between two engineers, between two contractors, or between a bond company and a contractor. Some of these unusual types are exceedingly difficult to draft properly, as, owing to their varying conditions, they cannot be systematized. It is mainly with them, therefore, that this lecture on contract writing is concerned; because for construction in general it is practicable to evolve a form which, when correctly filled out, will apply to any ordinary case.

FIG. 14. VIEW OF AUSTERLITZ BRIDGE DURING ERECTION, SHOWING SERVICE BRIDGE, FALSEWORKS, TRAVELERS, ETC.

The importance of drafting contracts properly cannot well be overestimated. An incorrectly drawn agreement is almost certain to involve serious trouble and often pecuniary loss to an innocent party; hence it behooves engineers to study thoroughly and fundamentally the science or art of contract writing.

Whether it be really a science or an art is a mooted point; but, in my opinion, the writing of proper specifications and contracts is certainly worthy to be termed a science.

Before one can draft a contract, he must have clearly in mind a full and well defined idea of all the conditions and desiderata, and he should epitomize these systematically before beginning to write. It is advisable to keep constantly in view the possibility that each party to the contract may be unscrupulous and willing to take every possible advantage of every weakness which the contract may contain and which will tend to his own profit—honor and integrity to the contrary notwithstanding. Failure to do this will often result in some ambiguity that will cause rank injustice to one of the parties to the agreement. It is difficult for an engineer to recognize this weakness of human nature and to bear it steadily in mind when writing contracts; because the training and the work of engineers tend to develop in them to an eminent degree the principles of absolute honesty; consequently, it comes hard for them to be forced to make a practice of doubting the integrity of their business associates. To mistrust the motives of one's fellow men is disagreeable but essential, if the writer of specifications and contracts is to protect himself or his clients from loss and fraud.

The essentials of a well drawn contract that comes within the province of the engineer, are as follows:

1st. A proper and customary form.

2d. A full and correct description of all parties to the agreement.

3d. A thorough and complete preamble.

4th. A statement of when and under what conditions the contract is to become operative.

5th. The limit, if any, for duration of contract.

6th. An exhaustive statement of what each party to the contract binds himself, his executors, administrators, successors, or assigns to do or to refrain from doing.

7th. A clearly defined enunciation of the consideration which each party is to receive—this is the essential raison d'etre of the instrument.

8th. The forecasting of all possible eventualities that would materially affect the agreement, and a full statement of everything that is to be done in case of each eventuality.

9th. Penalties for failure to comply with the various terms of the agreement.

10th. Provision for possible cancellation of contract.

11th. Provision for settlement of all business relations covered by the contract or resulting therefrom in case of cancellation, taking into account all possible important eventualities.

12th. Mention of the place where the agreement is drawn or of the place where it is to be put in force, so as to show the state under the laws of which the validity of the contract is to be determined, should suit be necessary to enforce it.

13th. Methods of payments, if any are to be made.

14th. Provision for extra compensation and the limitations connected therewith.

15th. Provision for possible changes in contract.

16th. Provision for transfer of the contract or for subletting.

17th. Provision for settlement of disputes.

18th. Provision for satisfactory and sufficient bond, if any be needed.

19th. Provision for defence of lawsuits, if such provision be necessary.

20th. Definition of names used in contract such as "Engineer," "Company," "Contractor," or "Trustee."

21st. Dating of contract.

22d. Proper signatures with the necessary seals, if the latter be required.

23d. Witnesses to the signatures, or execution before a notary public.

I shall now take up and discuss in the order of their enumeration each of these essentials to a properly drawn contract.

1st. The styles of opening clause for contracts are both numerous and varied, and it is difficult to say which is the best. Each writer naturally will have one favorite style and will adhere to it whenever possible.

Wait recommends the two following forms of introduction:

This Agreement, made and entered into this seventh day of February in the year 1905 by and between, etc., etc.

Articles of Agreement, made and entered into between The Kansas City Bridge and Terminal Railway Company, a corporation, etc., etc., and The Western Contracting Company, a corporation, etc., etc., on this eleventh day of February, 1905.

After the introductory clause comes the preamble, and immediately after it I insert the capital letters "NOW THIS AGREEMENT WITNESSETH," and follow with consecutively numbered clauses that embody all the terms and conditions of the contract, then close with provision for the signatures and seals of the contracting parties and witnesses to these signatures.

2d. In describing the various parties to an agreement, care should be taken to make the description full and convincing in order that there shall be no possible mistake concerning the identity of each party. This is effected in the case of an individual by stating his occupation and place of residence, in the case of a firm by naming it fully, mentioning its place of business, and describing the kind of partnership, and in case of a company by giving its legal title and the name of the state or country where it was incorporated. In case of a partnership it is sometimes well to specify whether it is general or special in respect to the work covered in the contract.

While most contracts are drawn between but two parties, it sometimes occurs that an agreement will involve three or even more. Such a contract is much more complicated and difficult to draft than one between two parties only.

Each party should be designated in the instrument by his special number, as the party of the first part or the party of the second part; and in addition it is well to give each another designation, such as "Contractor," "Company," "Owner," "Engineer," "Promoter," "Board," "City," "Incorporator," or "Trustee" in order to avoid the use of too many words throughout the document, as would be the case were he always referred to as the party of the first or second part. In order to make assurance doubly sure it is well in some cases to define the terms "Contractor," "Company," "Engineer," "Promoter," etc., at the end as well as at the beginning of the document. In any case these explanatory clauses should be placed at the beginning or the end of

FIG. 15. VIEW OF AUSTERLITZ BRIDGE DURING ERECTION, SHOWING TRAVELER.

the specifications, because the latter are often used without the contract being attached.

There is no strict rule as to the order in which the several parties shall be placed, but it is customary to make the one who pays the money the party of the first part. In case of employer and employee the employer should come first. In other cases it is a good rule to put the most important party first and the others as nearly as may be in the order of the importance of their relation to the enterprise or object matter of the agreement.

There is a consideration of primary importance in contract writing that is sometimes overlooked, viz., whether the parties to the agreement are legally entitled to enter into contract. For instance, in the case of a company, the president or general manager, or perhaps either, can sometimes legally contract in the company's name, but sometimes he cannot, in which case, if haste be essential, it would be proper to have him enter into and sign the contract and afterwards have it formally approved at a meeting of the board of directors. A properly certified copy of the board's approval should subsequently be attached to the contract. Access to its charter and by-laws is generally necessary to determine who has authority to enter into and sign contracts for a company.

In contracting, no corporation can exceed the limit of its powers as given by its charter. If it attempts to do so, its act will be ultra vires and without effect; consequently it behooves one in writing a contract with a corporation first to study well its charter, articles of incorporation, and by-laws.

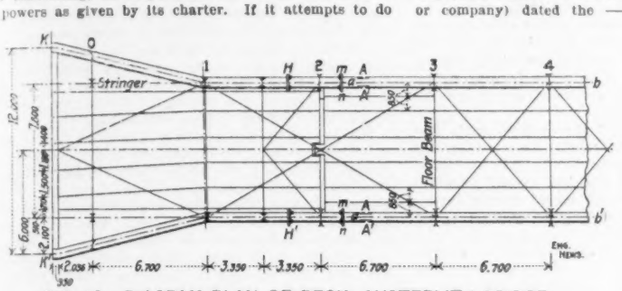


FIG. 13. DIAGRAM PLAN OF DECK, AUSTERLITZ BRIDGE.

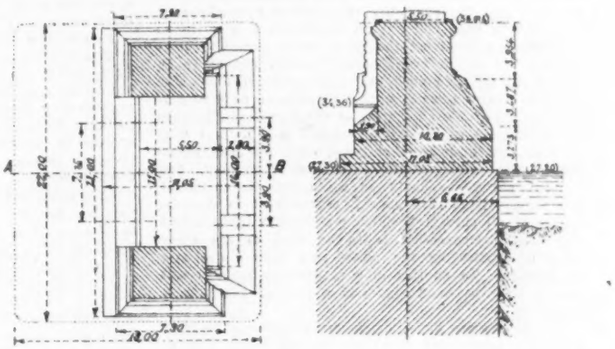


FIG. 16. PLANS OF ABUTMENTS FOR AUSTERLITZ BRIDGE.

Contracting with unincorporated organizations as parties, such as associations, clubs, societies, or congregations, is a precarious business; nevertheless it often has to be done. In order to ensure the payment of money obligations by such parties a sufficient sum should be deposited in advance in the hands of a reputable trustee with instructions to pay it to the proper party or parties as soon as the obligations covered in the contract have been met. Otherwise, the other contracting party is liable to lose his entire consideration, because it is very difficult to hold legally an organization that has no legal existence, even if all the members thereof be individually liable. Here again I speak from sad experience, for at the outset of my consulting practice I lost what I considered then a large fee by dealing with a committee of public spirited citizens, who were not honest enough to pay their just debts after the proposed enterprise had failed. Even the law did not enable me to collect the bill, as my lawyers did not present the case to the court in the proper manner.

Again, any person under twenty-one years of age, termed in law an infant, who enters into a contract, has the privilege of repudiating it after arriving at the age of maturity, in case that it does not redound to his advantage; consequently it behooves the writer of a contract to make sure in all doubtful cases that the contracting parties are of age. In engineering contracts, however, this question is seldom likely to arise because very young men are not often concerned in a prominent way with important enterprises.

Similarly, imbeciles, inebriates, and lunatics are incompetent, and contracts made by them are legally voidable at their option. While it is highly improbable that either an imbecile or a lunatic would ever be made a party to an engineering contract, it is not impossible that a man chronically addicted to the over use of liquor might be so concerned. Such a man might plead that he was under the influence of drink when he signed the document and thus possibly effect his release from its obligations, consequently the writer of an engineering contract should assure himself of the temperate character or at least of the sober condition of the parties thereto.

A married woman in some states cannot contract, sue, or be sued in her own name. While it is uncommon for women to be engaged in enterprises involving engineering, it is by no means impossible, as I have learned from a hard lesson; for in the case of a contract for the engineering of a large and novel enterprise that I entered into with a certain man, it transpired that he was acting as agent for a married woman. Before our work was finished the man died, and the woman gave us notice in writing that she would assume his share of the contract, and instructed us to finish our work. This we did, and she paid us one-half of our total fee; but before the date specified in the contract for the payment of the second half, the bottom dropped out of the scheme, and the lady then refused to make any further payment. She did not plead her married state as a justification for her refusal; but we knew what we might expect in a legal contest over the question, consequently we entered the balance on the wrong side of our ledger in the "Profit and Loss" account.

In case of war a contract entered into between parties who are subjects or citizens of the conflicting countries is illegal, and if war be declared subsequent to the signing of the contract, its obligations cannot be enforced by law until after the war has ceased. As engineers are often interested in projects in foreign countries, this is a matter that needs to be borne in mind when preparing the contracts for such enterprises.

When a contract is entered into by an agent, care should be taken to make this relationship both clear and legal in the document by stating the name of the owner or corporation and following it with the words "acting by and through Mr. X, Agent, Attorney, Engineer, President, or Treasurer (as the case may be), by virtue of the authority vested in him through power of attorney of the (here name the individual or company) dated the ——— day of ———, 19—, a copy of which is hereto annexed," or in some similar and equally explicit manner. In this way the name of the real principal is made certain, the authority of the agent is preserved, and the possible liability of the agent as the principal is averted. It must be remembered that no claims or obligations against a principal are created by a contract entered into by an agent who acts without proper authority, unless the contract be afterwards confirmed directly or indirectly by the principal.

Much engineering work is being done and is to be done in the future by contract with the United States Government. In making such contracts it is important to note that although the Government may enter suit on its contracts for their enforcement, it cannot, without its own consent, be sued for non-compliance therewith. Instances are not unknown of repudiation of contracts by governments. Furthermore, public officers cannot be held personally liable for contracts signed by them in their official capacity.

The names of the parties in the body of a contract should correspond exactly with the signatures and seals at the end, for a variation might prove fatal to the validity of the document.

3d. The preamble is a most important portion of any contract. It should explain fully all the whys and wherefores of the agreement and its raison d'etre. A thorough explanation of these would often render clear the intent of a clause in the body of the instrument that is otherwise ambiguous.

Once more I am speaking from sad experience, for in an important but hurriedly prepared contract one of the clauses was not drawn with sufficient clearness, and, in consequence, one of the parties to the agreement tried to take an unfair advantage of it. Had the preamble explained carefully and in detail the ultimate object of the contract and the various steps necessary for its accomplishment, the said party would not have been able to make the claim he did.

You will be better able to judge of the importance of the preamble and to understand its scope after hearing read the specimen contracts that are to conclude this lecture.

4th. Every contract should contain a statement of when or under what conditions it is to become operative. The date may be some particular day of month and year or immediately after or some definite time subsequent to some act or occurrence, such, for instance, as the giving of written notice, or the deposit of a certain amount of money in a certain place, or the completion of a certain piece of work, or the arrival of a railroad at certain point. Whatever the "condition precedent" may be, it should be made clear in the document beyond the peradventure of a doubt.

5th. Too often in contracts nothing is said concerning the duration of the agreement or of how it is to be drawn to a close. In some cases it would be impracticable thus to limit the life of the contract; but in others it is not only practicable but also advisable, and sometimes it is imperative, especially where a bond for proper completion of work is involved.

6th. The statement of what each party to the contract binds himself, his executors, administrators, successors, or assigns, as the case may be, to do or to refrain from doing should be thorough and complete in every detail. The importance of this is self-evident, nevertheless it is a point that is not always given proper attention in contract writing.

In all contracts between corporations or between a corporation and an individual, the promises to perform should be made binding upon the successors or assigns of each corporation, although it is probable that the law would enforce this even if the stipulation be omitted.

In contracts where an individual is a party to the agreement it is best to bind not only himself but also his executors or assigns, unless, perchance, the obligation be of such a nature as to be non-transferable, as, for instance, the performance of personal duties or services of an expert nature or involving special skill. Thus an engineer's services are not transferable, unless some special provision be made and agreed to by both parties that, in case of his death or inability for good and sufficient reason to finish his work, his contract is to be assumed by some other engineer either named or to be determined afterwards in some specific way. But the death of one member of a firm of engineers will not cancel an agreement; for as long as one of the original members of the firm remains in charge the contract will hold. In other words, it would require the death or incapacity of all the original members of the firm to abrogate the contract, unless special provision to the contrary exist in the written agreement.

Construction contracts are generally assignable, unless they contain provision to the contrary.

7th. The consideration which each party to an agreement is to give and is to receive should be clearly and fully stated in the document, otherwise unsealed contracts are liable to be held valueless and void in law. Moreover, the consideration must be real, substantial and adequate. Some lawyers make a practice in many cases of specifying a consideration of one dollar, and they even try to pass that dollar around among the several parties to the agreement by having each party make nominally that payment to each of the other parties so as to show that each receives a valuable (?) consideration. In my opinion, such a practice is mere humbug and unworthy of adoption by any man pretending to scientific attainments in his profession, no matter whether that profession be law or engineering. Its adoption, it seems to me, is prima facie evidence of weakness in the document and a confession by its writer that he has failed to make evident the true consideration that each party is to receive and the real reason for each party's entering into the agreement.

There may be some excuse for passing the dollar in case of a parent deeding property to his child, where the true consideration is love and affection; but a dollar does not constitute a real consideration—it would be insufficient usually to pay the cost of typewriting the document, hence its employment is a fiction and a farce.

8th. No portion of the work of contract writing requires greater experience and ability than the forecasting of all possible eventualities that would materially affect the agreement and the proper provision for what is to be done in the case of each eventuality. All contracts are more or less faulty in this particular, for it would require omniscience to forecast all future happenings; nevertheless, in preparing an important contract one should endeavor to foresee and provide for all possibilities and probabilities. The lawyer or engineer who makes a practice of giving this important matter full consideration in every contract that he writes will soon find himself in demand by capitalists to aid them in making their investments and in consummating their enterprises. The succeeding examples

of contracts will illustrate what I mean by the forecasting of eventualities and providing for them.

9th. The matter of penalties is one that has to be handled with gloves, for the law is very jealous of its rights and prerogatives, and deems that it alone is authorized to specify and enforce a penalty, which it interprets as a punishment for failure to perform or comply with the terms of an agreement. On this account it is better not to use the term "penalty" in any contract but to employ instead that of "liquidated damages." In my practice I have a clause in construction specifications that reads as follows:

For each day of delay beyond the date set in the contract for completing the entire work herein outlined, all in accordance with the plans, specifications and directions of the Engineer, the Company shall withhold permanently from the Contractor's total compensation the sum of ———— dollars; and the amount thus withheld shall not be considered as a penalty, but as liquidated damages, fixed and agreed to in advance by the contracting parties as a proper compensation to the Company for the loss caused it by such delay.

Liquidated damages are but seldom enforced, owing mainly to the characteristic good nature of engineers; for they object to taking advantage of a contractor who has worked faithfully but has been unfortunate. Again, the fact that the sympathy of jurors is generally with the working man and against corporations is a reason why disputes involving the retention of money to compensate for delays are generally settled out of court.

10th and 11th. In most contracts for construction and in some other types of contract there is no need to provide for a possible abrogation of the agreement, because the completion of the work involved is a natural cancellation; but in some other types, such, for instance, as partnership contracts that continue indefinitely, full detailed provision should be made for annulment at any time. Great care should be exercised to describe fully how all current business matters are to be closed and what compensation is to be paid to the other party or parties by the party who desires the said cancellation. To do this in a satisfactory manner will require business knowledge and ability of the highest order.

12th. It is quite important in many contracts to state where the instrument was executed and where it is to be put in force, notwithstanding the fact that the residence of each party in case of individuals or the state of organization in case of corporations has been described in the introductory clause of the document. The laws governing a contract may be determined by the place where the contract was made or by that in which it is performed. Wait treats this question very thoroughly on pages 49 to 51 of his "Engineering and Architectural Jurisprudence."

13th. Methods of making payments under construction contracts are generally covered in the specifications, where, in my opinion, they properly belong, although I have on several occasions been adversely criticized for not putting a payment clause in the contract proper. In all other types of contract in which payments of money are involved, full provision should be arranged for the exact manner in which all payments, both partial and final, are to be made. This remark applies with special force to contracts involving engineering fees; for in these, if payments on account are not arranged for, there is a chance that the engineers will receive no compensation at all until after the completion of their work, and this might be delayed for an indefinite period. Our usual practice is to ask one-half of our fee upon the completion of the plans and specifications and the other half in monthly payments proportionate to the amount of contract work done on the construction, so that when the latter is finished we shall have been paid in full. We have learned to provide also that we are compensated properly for all extra expense to us due to failure to complete the work in the time specified. It has cost us many thousands of dollars to learn this lesson, consequently the hint that I am giving you ought to be valuable.

14th. In construction contracts the subject of extra payments also belongs in the specifications, although in many cases it is covered in the contract proper. Our standard clause for this item reads thus:

No extras will be allowed, unless they be ordered in writing by the Engineer. For extras so allowed the Contractor will be paid the actual cost to him plus 10% for profit. Satisfactory vouchers will be required from the Contractor for all extra labor and materials.

15th. It is a wise precaution to provide for making changes in every important contract. Our standard clause for this item is as follows:

No change or alteration shall be made in the terms or conditions of this agreement without the consent of both parties hereto in writing; and no claim shall be made or considered for any extra work, unless the same shall be authorized and directed in writing by the Engineer.

16th. In construction contracts there should always be a clause to govern assigning the contract and sub-letting the work. Our standard clause for this reads thus:

The party of the second part hereby agrees that it will not assign or sub-let the work covered in this contract, or any portion of it, without the written consent of the party of the first part; but will keep the same within its control.

17th. In respect to provision for settlement of disputes engineers are somewhat at variance. Some think that the Engineer should be the sole arbiter, but I do not believe that such an arrangement is just, savoring, as it does, altogether too much of autocratic rule. I am a firm believer in arbitration for the settlement of all disputes on important matters, and it has for many years been my custom to provide in all construction contracts for such a method of settlement. Our standard clause for this matter is as follows:

The decision of the Engineer shall control as to the interpretation of drawings and specifications during the execution of the work under them; but if either party shall consider itself aggrieved by any decision, it may require the dispute to be finally and conclusively settled by the decision of three arbitrators, the first to be appointed by the party of the first part, the second by the party of the second part, and the third by the two arbitrators thus chosen. In case that the two first chosen fail to agree upon a third, the latter shall be appointed by .. .. ..

the said second party to proceed, and continue, and complete the same, as if such time had not elapsed, such permission shall not be deemed a waiver in any respect, by the first party, of any forfeiture or liability for damages arising from such non-completion of said work within the time specified, and covered by the "Liquidated Damages" clause of the specifications but such liability shall continue in full force against the said second party, as if such permission had not been granted.

Eighth.—No change or alteration shall be made in the terms or conditions of this agreement without the consent of both parties hereto in writing; and no claim shall be made or considered for any extra work, unless the same shall be authorized and directed in writing by the Engineer.

Ninth.—In the event of any delay in completing the work embraced in this contract, the party of the second part shall be entitled to no extra compensation on account of such delay; as it is hereby assumed that in submitting its tender it took its chances for the occurrence of such delay. If, however, in the opinion of the Engineer, the Contractor be delayed by any act of the Company to such an extent as to cause him serious hardship, such as a temporary cessation of the work, the Company shall allow the Contractor whatever compensation for such delay as may appear to the Engineer to be just and equitable.

Tenth.—The party of the second part hereby agrees that it will not assign or sublet the work covered in this contract, or any portion of it, without the written consent of the party of the first part; but will keep the same within its control.

Eleventh.—The decision of the Engineer shall control as to the interpretation of drawings and specifications during the execution of the work under them; but if either party shall consider itself aggrieved by any decision, it may require the dispute to be finally and conclusively settled by the decision of three arbitrators, the first to be appointed by the party of the first part, the second by the party of the second part, and the third by the two arbitrators thus chosen. In case that the two first chosen fail to agree upon a third, the latter shall be appointed by .....
By the decision of these three arbitrators, or by that of a majority of them, both parties to this agreement shall be finally bound.

Twelfth.—As, according to the terms of the accompanying specifications, which form a part of this contract, the party of the second part is to indemnify the party of the first part against all liability or damages on account of accidents occasioned by the omission or negligence of itself, its agents, or its workmen during the continuance of this agreement, and against all claims for royalties on patents; it is hereby agreed that the party of the second part shall be promptly and duly notified in writing by the party of the first part of the bringing of any such suit or suits, and shall be given the privilege of assuming the sole defense thereof. The party of the second part is to pay all judgments recovered by reason of accidents or patents in any suit or suits against the party of the first part, including all legal costs, court expenses, and other like expenses.

Thirteenth.—The Contractor, further agrees to give to the Company a surety-company bond, satisfactory to the party of the first part in the sum of .....
for the faithful performance of this contract and the specifications, and of all the terms and conditions therein contained, and for the prompt payment for all materials and labor used in the manufacture and construction of the structures, and to protect and save harmless the company from claims on patents and from all damages to persons or property, caused by the negligence or claim of negligence of the Contractor, his agents, servants, or employees in doing the work, or in connection therewith, and from injury to or loss of materials paid for by the company either partially or in full before the completion and acceptance of the construction or constructions.

Fourteenth.—The word "Engineer" as used in this contract refers to the Consulting Engineers of the .....
.....
or their duly authorized representative.

IN WITNESS WHEREOF, the parties to this agreement have hereunto set their hands and seals.

Dated the day and year first herein written.

WITNESSED BY

.....
.....
.....
.....

The ninth item of this form, as originally written without qualification, has been severely criticized on more than one occasion because of alleged unfairness. I refer to the clause which reads:

> In the event of any delay in completing the work embraced in this contract, the party of the second part shall be entitled to no extra compensation on account of such delay; as it is hereby assumed that in submitting its tender it took its chances for the occurrence of such delay.

I admit the onesidedness of this clause and that if it were enforced to the letter by a narrow-minded or unfair engineer, it might effect a hardship upon the Contractor; nevertheless, as a matter of policy we still leave it in our construction contracts in order to prevent contractors from constantly making claims for extras on account of trivial delays; but, if a delay were of such importance as to necessitate a partial or total shutting down of the work, we would certainly see that the Contractor is compensated for the extra expense to which he is put by stopping and re-starting construction, for in our dealings with contractors we aim always to act the part of "judge" and not that of partisan of the company.

Once in a while it becomes necessary for a Company to let work on the basis of actual cost plus a percentage for profit, and in this case special care has to be taken to cover all possible details and eventualities in order to protect the Company from extortion and needless expense. This manner of letting work is very favorable to the Contractor, but is troublesome and expensive for the Engineer and rather risky for the Company. Under certain conditions, however, it is the only way in which the work can be let at any reasonable figure, such conditions, for instance, as govern railroad or bridge construction in a tropical country where fevers are prevalent and where the conditions affecting the cost of construction are practically unknown. My firm has acted as engineers on such work, and, consequently, we have learned by experience how contracts therefor should be prepared. The following is the type of contract that we would draft for such a case.

Let us assume that there is a railroad being built in the tierra caliente of Mexico from Matamoras on the American border through or near Tampico and Vera Cruz to the border of Guatemala, by an American Company organized in West Virginia under the name of the Mexican Gulf Coastline Railway Company, and that it desires to let to the Central Bridge Company of Kansas City, Missouri, the contract for building the substructures of all bridges and all the culverts on the line and for erecting all the superstructures, the metal for which is to be furnished and delivered at the bridge sites by the Company. Let us assume also that the basis of the contract is that the Contractor is to be paid the actual cost to him of the entire work plus fifteen (15) per cent. for profit.

Under such conditions we would prepare specifications of our standard type, with possibly a few modifications, and would attach them to the following contract:

MEMORANDUM OF AGREEMENT, made and signed this fourteenth day of March, 1905, by and between the Mexican Gulf Coastline Railway Company, a corporation of the State of West Virginia, U. S. A., and having its principal business office in New York City, N. Y., the party of the first part, and sometimes termed in this agreement and in the specifications the "Company" and the Central Bridge Company, a corporation of the State of Missouri, U. S. A., and having headquarters at Kansas City, Mo., the party of the second part, and sometimes termed in this agreement and in the specifications the "Contractor."

WHEREAS, The party of the first part has already begun the construction of its road at several points along its line, which starts from the American border at Matamoras, Mexico, and parallels the Gulf coast running near the cities of Tampico, Tuxpan, and Vera Cruz, and extending to the border of Guatemala, and

WHEREAS, the party of the first part has already entered into contract with certain American firms for the manufacture of the metalwork for the superstructure of all the bridges required for its entire line, and

WHEREAS, The party of the first part desires to let to some reliable American contractor the contract for building all the bridge piers, abutments and culverts and for erecting all the superstructures of bridges on the entire line of its road, and

WHEREAS, The party of the second part makes a specialty of building the substructures and superstructures of bridges, and has had a wide experience in this line of construction,

NOW THIS AGREEMENT WITNESSETH:

First.—The party of the second part, for and in consideration of certain payments to be made to it as hereinafter specified, will furnish all the materials for and construct complete all the bridge piers, abutments, and culverts, will erect and paint all the metalwork for the superstructure, and will furnish and put in place all the flooring for the bridges of the entire line of the said railway, all in accordance with the specifications, hereunto annexed and made a part hereof, and with the plans and directions of the Engineer.

Second.—The party of the second part shall commence the work of construction as soon after the signing of the contract as it is practicable to begin, starting at as many different points along the line of the road as the Engineer may direct, and shall push the same to completion as rapidly as possible.

Third.—In consideration of the performance by the party of the second part of its covenants and agreements, as hereinbefore set forth, the party of the first part hereby covenants and agrees, for itself, its successors, and assigns, to pay to the party of the second part for the furnishing and completion of the entire work the actual cash cost thereof plus fifteen (15) per cent. for profit.

In computing the cost of the work, there shall be included all items of materials, labor, and transportation of men, materials, and plant to and fro, but no allowance will be made for cost of plant or deterioration of same, or for the time or personal expenses (other than railway and Pullman fares) of the party of the second part, or for interest on money required to carry on the work. Accident insurance for employees, insurance against loss of materials or plant by fire or shipwreck, and all stamps for documents and taxes of all kinds shall be considered items of legitimate expense, and shall be allowed for by the Engineer in the monthly estimates, as shall also the cost of housing the workmen, but not their sustenance.

By the term "Plant" is meant all machinery and apparatus, new or second hand, such as engines, dredges, diving apparatus, pile drivers, wire rope used for tackle, blocks, forges, riveting apparatus, and blacksmith's tools that are of a permanent nature; but does not include such perishable materials as hemp rope, rubber hose, wheelbarrows, bolts, shovels, rubber boots, and all other tools and apparatus of a temporary nature. The decision of the Engineer shall control as to what is and what is not "Plant," and his decision shall be final.

In respect to repairing plant and tools it is understood that all minor repairs that are done by the blacksmith and other employees of the Contractor are to be charged to the Company, as are also repairs during construction to the perishable parts of the plant and tools, such for instance, as the timber for pile drivers, the easily broken parts of riveting machinery, and pipes for boring outfit; but when the work of the Contractor is finished there will be no allowance made to him for the deterioration of his plant, nor will he be allowed to put it into thorough repair at the expense of the Company just before he completes the entire construction. When this completion occurs all plant is to be the property of the Contractor, and all other apparatus of any value and all surplus materials are to be delivered to the Comnapy as per the directions of the latter. But the Company reserves the right to purchase from the Contractor any of his plant at its actual value where it was bought less a proper allowance for depreciation; and the Engineer is to be the sole arbiter concerning the actual value of such plant in case that the Company and the Contractor fail to agree thereon.

No sick or incapacitated employee shall be included on the Contractor's pay-roll without the written consent of the Engineer in each and every case.

In respect to traveling expenses for employees, none shall be paid for the journey to the work until after the said employee has been on the work for three (3) months; and no return expenses shall be allowed unless the employee has been discharged on account of no fault of his own, or until he has worked for the Company at least twelve (12) months. The traveling expenses allowed shall cover nothing but the railroad fare, except that in the case of the Contractors, and their regularly salaried employees sleeping car charges, also shall be borne by the Company.

The men's time spent in traveling from the U. S. to their work is to be paid for in case they remain three (3) months or more continuously thereon; otherwise there will be no allowance for time spent in traveling. In no case will any workman be allowed compensation for the time spent in returning home. No employee discharged for incompetency or misbehavior shall be allowed any return traveling expenses.

If, during the progress of the work, in the opinion of the Engineer, its prosecution is being delayed by reason of these restrictions concerning traveling expenses, the said restrictions may be modified or removed altogether by the mutual consent in writing of the two parties to this agreement, but in this manner only.

No material is to be paid for until after it is delivered at site, unless the Company shall have had it in its possession for over thirty (30) days.

Medical attendance and medicine at site will be paid for by the Company; but no other medical or hospital expenses will be allowed; and the Company will bear all expenses connected with providing boiled and filtered or condensed water for drinking purposes.

The commissariat department shall be considered as a thing apart from this contract. The Contractor shall manage it at his own expense, and shall charge enough for board to reimburse himself for the entire outlay connected therewith and to provide a profit not exceeding fifteen (15) per cent. At the same time the Company shall insist that all employees be properly fed. The Company will make no charge for hauling commissariat supplies for the Contractor over its own lines. In respect to the proper feeding of employees and a reasonable price for board, the Engineer shall be the sole arbiter in case that any dispute arise; and to this end the Contractor's commissariat books shall always be open to his inspection.

In order to keep the accounts straight, all payments made in Mexico for labor and materials are to be in Mexican money, but the Engineer's monthly estimates are to be made in American money, the rate of exchange to be adopted in effecting the reduction being that ruling in the City of Mexico on the date of the Engineer's estimate.

All bills for materials purchased outside of the Republic of Mexico, exceeding in amount five thousand dollars ($5,000.00) in gold, after being approved by the Engineer, shall be sent directly to the Company for payment; but the Contractor shall receive his fifteen (15) per cent. profit upon them.

Tools and other supplies may be furnished the Contractor by the material department of the Company, in which case the Contractor shall receive his fifteen (15) per cent. profit on the wholesale cost to the Company of any materials upon which profit would be allowed, were they purchased elsewhere.

The Contractors' books shall at all times be open to the inspection of an expert accountant or accountants appointed by the Company to investigate the accounts.

The methods and times for payments shall be as described in the accompanying specifications, which form a part of this contract.

Fourth.—The number of men to be sent from the U. S. A. to the work and their salaries or wages shall be subject to the approval of the Engineer, and he shall be satisfied as to their fitness before they are engaged. The number of men to be employed at each site shall also be subject to the Engineer's approval. He may order that the number be increased if, in his opinion, there are not enough, or that it be decreased, if it be too great for economic working.

The Contractor shall purchase plant and materials in advance of their requirement, so as to have no idle men on the work. In case that he fail to do so, the Engineer shall have the right to prevent the Company from loss of money because of such negligence, by making an equitable reduction in the monthly payments.

The Engineer shall approve the purchase of all materials obtained outside of the Republic of Mexico and all important purchases of materials in same. The decision as to what purchases are important and what are not shall rest with the Engineer.

All expense bills are to be properly vouchered in triplicate before being presented to the Engineer for his approval, and none shall be honored by the Company without his approval. The Engineer shall attach one set of vouchers to his monthly estimates, shall keep another set for his own records, and shall return the third to the Contractors.

All pay-rolls are to be signed by the payees in the presence of the Engineer or one of his duly authorized representatives; and when they are complete they shall be copied in duplicate in typewriting and the copies shall be attested to by a notary. One attested copy is to be attached to the monthly estimate, the other is to be retained by the Engineer, and the original is to remain the property of the Contractor. In case of employees unable to write, the paymaster shall attach to the pay-roll an affidavit certifying that all the men on the said pay-roll have been paid in full.

The Company is not to be held liable for the failure of the Contractor to comply with any of the Mexican laws, especially those relating to books, accounts, and stamping of documents.

Before the plant is shipped to the site of the work it must be approved in writing by the Engineer; and the Contractor shall provide in advance whatever plant the Engineer may deem necessary for a satisfactory prosecution of the work, including several complete outfits for pneumatic riveting.

Fifth.—All material paid for by the party of the first part shall be deemed to have been delivered to and to have become the property of the said first party; but the party of the second part hereby agrees to store it and to become responsible for it during the continuance of this agreement or until it has been placed in the work and accepted by the Engineer. If it be possible, all materials that are liable to injury by fire are to be insured to as near their full value at the place where stored as is practicable. In case that the Contractor fail so to insure and any of the Company's property that is in his charge be injured or destroyed, it shall be repaired or replaced according to the directions of the Engineer, and the cost of such repairs or replacements are to be deducted by the Engineer from the Contractor's monthly estimates. In case it be impossible to insure the said materials, the Company will stand the cost of all loss by

fire, provided that, in the opinion of the Engineer, the Contractor used due diligence in caring for the materials so injured or destroyed; but the Company shall in no way be liable for injury to or loss of materials by water, flood, theft, or maliciousness.

Sixth.—The party of the first part shall secure to the party of the second part the right to enter upon the places to be occupied by the structures and by the materials thereof during construction, free from cost, damage, or claim for damage of any kind whatever.

Seventh.—No change or alteration shall be made in the terms or conditions of this agreement without the written consent of both parties hereto.

Eighth.—The party of the second part hereby agrees that it will not assign or sublet the work covered in this contract or any portion of it, without the previous written consent of the party of the first part; but will keep the same within its control. It will not be the policy of the Company to permit any sub-letting by the Contractor, because its principal object in letting this contract for cost plus a percentage for profit is to avail itself of the said Contractor's experience, constructive skill, and ability to handle men and work.

Ninth.—If the Engineer should find occasion to reject and insist upon the removal and replacement of any portion of the completed or partially completed work, and if, in his opinion, the Contractor is to blame for the defects, the cost of such removal and replacement is to be deducted by the Engineer from the Contractor's monthly estimates.

Tenth.—If, during construction, it appear to the Engineer that the Contractor is not making proper progress on account of insufficient plant, labor, materials, supplies, or energy, the Company shall give the Contractor notice in writing that it is not satisfied with the progress that is being made, explain its conception of the reasons for the delay, and suggest remedies therefor. Then if, after the expiration of twenty (20) days, the Engineer report that the Contractor has not taken the steps indicated by the Company as necessary for a satisfactory prosecution of the work or other steps which, in the opinion of the Engineer, will prove equally effective, the Company shall have the right, after giving the Contractor a further twenty (20) days' notice in writing, to undertake itself, either by administration, or by letting the contract to other parties, the completion of the said work which is thus being neglected.

Under these circumstances the Company shall have the right to enter upon and take possession of the plant, tools, materials, and supplies of the said Contractor, or any part thereof; and in such case the Contractor shall be allowed a fair price for the use of all plant returned, and compensation in full for any portion thereof used up or expended on the work.

This provision shall apply to the entire work involved in the contract as well as to any portion of it; which means in effect that if, in the opinion of the Engineer, the Contractor does not make a practice of prosecuting his work with due diligence, thoroughness, or economy, the Company shall have the right to finish the said work in the manner provided in this clause of the agreement.

Eleventh.—The decision of the Engineer shall control, etc., etc. [This clause is taken without change from Waddell and Hedrick's standard form previously quoted.]

Twelfth.—[This also is from our standard form and covers the question of defence of lawsuits.]

Thirteenth.—Whenever the reserve of ten (10) per cent. of the monthly estimates, provided for in the contract, amounts to fifty thousand dollars ($50,000.00) the Company shall pay to the Contractor twenty-five thousand dollars ($25,000.00) on account, so that at all times after the first quarter million dollars' worth of construction is allowed for in the estimates, there shall always be a reserve in the Company's hands varying in amount between twenty-five thousand (25,000) and fifty thousand (50,000) dollars.

Fourteenth—The Contractor further agrees to give to the Company a good and satisfactory surety-company bond in the sum of fifty thousand dollars ($50,000.00) gold for the faithful performance of this contract according to the specifications, and for the prompt payment for all material and labor used on the work, and to protect and save harmless the Company from all damages to persons or property caused by the negligence, or claim of negligence, of the Contractor, or his agents, servants, or employees in doing the said work or in connection therewith, the form of the said bond to be satisfactory to the Company. In case though, on account of the construction being done in a foreign country, there be experienced special difficulty or expense in obtaining a satisfactory surety-company bond, the Contractor shall deposit with some Trust Company of New York City, to be named by the party of the first part, securities to the full value of fifty thousand dollars ($50,000.00) as a guarantee instead of the aforesaid bond. Whatever surety be decided upon, it shall continue in full force during the existence of this contract; and it shall not be waived or voided by any change in the plans, specifications, or amount of work involved, nor by any change in the contract itself that, in the opinion of three arbitrators (one appointed by the surety-company, one by the party of the first part, and the third by the two arbitrators thus chosen), is not in equity a good and sufficient cause for the cancellation of the surety. All these conditions will have to be agreed to in advance by the proposed surety company, or else its surety will not be considered by the party of the first part; and in case of a cash or equivalent guarantee, the party depositing the same will have to agree to the said conditions.

Fifteenth.—In case of any arbitration, if the two arbitrators first chosen refuse or fail to decide upon a third, the latter shall be appointed by the Mayor of New York City.

Sixteenth.—The word "Engineer" as used in this contract refers to the Consulting Engineers of the Mexican Gulf Coastline Railway Company or their duly authorized representatives.

Seventeenth.—Either party to this agreement shall at any time have the right to make this contract a public contract according to the laws of the Republic of Mexico.

Eighteenth.—In case that at any time the Company decides to discontinue either temporarily or permanently the construction of its railroad or bridges, it shall have the right either to order the Contractor to cease operations until further notice or to cancel the contract. In the first case the Company shall pay all the expenses for closing down, storing, and taking care of the plant, and starting work again, also what, in the opinion of the Engineer, is a proper allowance for interest on cost of plant while it is out of use, together with fifteen (15) per cent. on these amounts for profit; and in the second case the Contractor is to be paid in full for all work done and materials furnished up to the date of cessation besides the entire cost of transporting the men and plant back to Kansas City, U. S. A., with the usual fifteen (15) per cent. added for profit.

IN WITNESS WHEREOF, the parties to this agreement have, in the City of New York, hereunto set their hands and seals.

Dated the day, month, and year first herein written.

Attest.

....	The Mexican Gulf Coastline Railway Company.
Secretary of the Mexican Gulf Coastline Railway Company. [Seal]	By ..... President.
....	The Central Bridge Company.
Secretary of the Central Bridge Company. [Seal]	By ..... President.

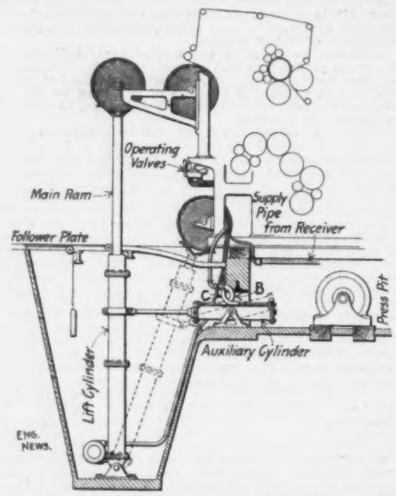
You may have noticed that while this contract contains many clauses that are not included in the ordinary type of construction contract, the latter has certain provisions that do not exist in the former, notably a clause for "liquidating damages." The reason for this omission is that under the assumed conditions it was impossible to set any dates for the completion of the various structures or of the entire work involved, hence no penalty could be demanded.

I desire to call your attention to a feature of this method of letting work on a percentage basis that is, perhaps, foreign to the subject of the lecture, viz., the unusually large amount of labor that it involves for the engineers, who, in addition to their ordinary duties, have practically to assume the functions of administrators. For this reason their fees on such work should be greater than on ordinary contract construction.

A SPECIAL AIR LIFT FOR PRINTING PRESS SERVICE

By A. W. Cochran.*

With the installation of the large newspaper perfecting presses of to-day, necessary to meet the enormous circulation of the metropolitan daily, it becomes apparent that some efficient method of handling the large rolls of paper must be devised. As often as fresh paper is supplied to a press, the roll of paper, weighing about 1,000 lbs. on the average, must quickly and safely be lifted into the journals of the roll-carrying brackets. When it is understood that a few of the large dailies consume about 200 tons of paper each week, and that a portion of this amount must, generally, be lifted two and three decks high, it will be admitted that the duty of a suitable lift, whether electric, pneumatic or hydraulic, is rather severe.



Air Lift for Supplying Paper to a Newspaper Press; in Press-Room of Oregonian Publishing Co., Portland, Ore.

Recently, at the time a new sextuple press was installed at Portland, Oregon, in the newspaper plant of Mr. H. L. Pittock, publisher of the "Morning Oregonian" and "Evening Telegram," several devices were offered for the service above described. The local conditions, however, eliminated most of the commercial devices. The overhead space was too low for an electric drum hoist, as the pressroom was only 13 ft. 2 ins. high, while the roll for the upper deck had a lift of 7 ft. 9 ins. The pneumatic hoist offered no better solution of the difficulty, since it required more headroom than the electric. The ordinary hydraulic sidewalk lift with a small platform where the paper could be rolled on at the basement floor line and rolled off on the upper brackets, seemed to offer some possibilities. But with the hydraulic device came the necessity of installing a separate pumping plant and piping under the press, which involved the hazard of leaky pipes jeopardizing the composition rollers and other finished parts of the press equipment. This, together with the fact that the lift could be used only for the upper roll, leaving the lower roll to be lifted into place by hand, caused a reconsideration of the plans and resulted in the design of a special air lift, shown in the sketch herewith.

This lift is supplied with air at a pressure of 85 lbs. per sq. in. from a 4 ft. × 5 ft. receiver hung in strap-iron stirrups from the pressroom ceiling. An Ingersoll-Sergeant "Class E" compressor, driven by a 13-HP. Westinghouse motor through a Morse Silent-Chain drive, keeps this receiver charged, under control of a pressure-regulator and a Cutler-Hammer automatic motor-starter.

Air is carried from the receiver through the supply pipe shown in the sketch to the operating valve, mounted in convenient handling position on the back of the press. This valve is double, consisting of two valves of taper-plug construction, whose lever handles lock centrally when not in use, thus avoiding starting from accidental contact. One of the valves commands a pipe, A, leading to the bottom of the main cylinder, and can either admit air to the cylinder, to raise the main lift, or exhaust from the cylinder, to lower the main lift. The other valve controls two pipes, B and C, leading to the two ends of a horizontal cylinder which is arranged to swing the main lift about a pivot at the bottom, thereby moving its carriage toward or away from the roll brackets of the press.

The main cylinder is pivoted at the bottom on a 3-in. shaft resting in suitable trunnion castings. The ram of this cylinder is built up of two 5-in. channels placed flanges together, with separators between at intervals of 14 ins., the whole being riveted together with ⅜-in. countersunk rivets. It is guided at the top of the cylinder by bronze guides. The ram has proved itself to be quite stiff, showing very little deflection under full load at the top of the stroke. At the top of the ram is a V-shaped platform, made of ⅜-in. boiler steel connected to the ram by heavy wrought-iron forgings.

The horizontal auxiliary cylinder is pivoted at the middle of its length in horizontal trunnion bearings, whose supports, like those of the base pivot of the main cylinder, are anchored to the concrete foundations by expansion bolts. A double-cupped piston works in this cylinder; its rod passes through a stuffing-box in the front cylinder-head, and at its forward end has a pin attachment to a ring clamped on the main cylinder near its upper end.

The valve ports controlling the pipes B and C of the auxiliary cylinder are so arranged that when compressed air is admitted to one end of the cylinder, through one of these pipes, the other end exhausts to atmosphere; or, both ports may be closed, to hold the cylinder in any desired position. By proper proportioning of these ports, it was found, a cushioning effect could be obtained which made the motion of the auxiliary piston very gentle. The tendency of the main lift to run in toward the press when loaded and in inclined position is completely neutralized, and the load is at all times under perfect control. The two valves can be operated simultaneously or separately, so that the roll of paper can be landed quickly and easily in either set of roll brackets.

As shown in the sketch, the upper roll brackets are long enough to provide storage for one roll while the other is being fed into the press, so that there is minimum delay in changing paper. By lifting the empty roll shaft to the back end of the brackets, it is in place to be brought to the floor by the lift to be put into a new roll.

When the lift is not in use it can be drawn out of the way under the lower paper roll, leaving the floor behind the press clear for passage. One serious objection turned up when the lift was in-

*Operating Engineer, Oregonian Publishing Co., Portland, Ore.

stalled, in the shape of a 3¾-in. slot in the floor the entire length of travel of the main ram. This objection was met by putting in a follower plate with weight attached, arranged to cover the slot in the floor as the ram was drawn in toward the machine.

This lift has been in operation for about five months, and has proven to be very reliable and effective for the work required. It was designed and installed by the Mechanical Department of the Oregonian Publishing Co., and was built by the Portland Iron Works, of Portland, Ore.

AN OPEN-DRUM CONCRETE MIXER.

The accompanying cut represents a concrete mixer invented by Mr. J. W. Page, and owned by Page & Shnable, contractors, of Chicago. It is of the batch type, with an open drum so that the concrete is always in view of the engineer or inspector during the process of mixing. The drum is mounted on a shaft inclined at an angle of 52° from the vertical, and having at its lower end a bevel wheel gearing with a pinion on a shaft having a spur wheel driven by a pinion on the engine shaft. In the larger sizes a reversible engine is used, so that if the stone shows any tendency to separate from the other material the drum can be reversed and thus effectually complete the mixing. There are no blades or wings in the drum, the mixing being effected by the materials being carried up the slope by the revolution of the drum and continually sliding and rolling down. When the mixing is completed, a door in the side is opened and the batch falls into a car or wheelbarrow below. The machines are made in sizes to handle batches of ½ to 1¼ cu. yds., and Fig. 1 shows a small portable machine with mixer, engine and boiler complete, all mounted on a frame upon wagon wheels.

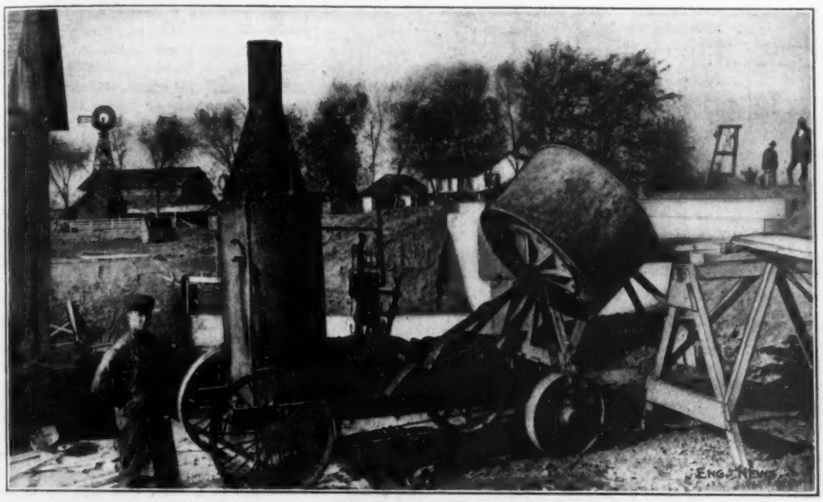


FIG. 1. A PORTABLE CONCRETE MIXING PLANT.
Page & Shnable, Chicago, Builders.

A large machine of this type is in use at the power plant of the Chicago drainage canal near Lockport, Ill. This makes batches of 1¼ cu. yds. of concrete, and has a drum 8 ft. diameter and 4 ft. deep, driven by a two-cylinder vertical, reversing engine of 20 HP. Steam is piped from a central boiler plant, and a water pipe is carried over the mixing platform, with a branch projecting into the drum. The drum makes about 12 to 15 revolutions per minute, and a complete batch can be mixed in 2 to 2½ minutes. In addition to this work, the machines have also been used on the Pecos Valley concrete arch aqueduct, the Hennepin canal locks, and the dam and water power development of the Oliver Plow Works at South Bend, Ind.

NOTES FROM THE ENGINEERING SCHOOLS.

UNIVERSITY OF MICHIGAN.—The teaching force of the forestry department has been increased by the appointment of Mr. Walter Mulford as assistant professor of forestry. Professor Mulford received the degree of B. S. in Agriculture at Cornell in 1899, and the degree of Forest Engineer at the same institution in 1901. Since then he has been for three years Forester of the Connecticut Agricultural Experiment Station at New Haven, and for the year 1904-5 he was in charge of commercial tree studies in the Southern Appalachian region, for the U. S. Bureau of Forestry.

THE FIRST ATTEMPT TO MANUFACTURE NATURAL CEMENT IN THE UNITED STATES.

By Edwin C. Eckel, C. E.*

The date of the first manufacture of natural cement in this country is usually stated as 1818, the place of the enterprise being in Central New York, along the line of the Erie Canal. So far as actual productive results are concerned, this statement is correct enough, but in looking up contemporary accounts of this early work I have recently come across a very interesting letter describing a still earlier attempt to manufacture this product by a Mr. Randolph, of Virginia. The Virginia experiments were carried on early in 1817, antedating the New York manufacture by over a year.

The account of his experiments and results is contained in a letter from Randolph to Dr. Samuel Latham Mitchill, of New York, who was then easily the leading scientist of the country. The letter is dated Richmond, Va., June 26, 1817, and is as follows:

Dear Sir: Among the great variety of useful inventions connected with the arts, as it has been lately my object to pursue and to have observed in England, it was one of great importance in my mind, to discover in this country something that might answer an equally valuable purpose with the famous "Dutch Terias," or Parker's "Roman Cement," as it is now generally used in England and the West Indies for works under water and elsewhere situated.

In this pursuit it has been my good fortune to succeed so far as to indulge a confidence that upon the more satisfactory tests of works upon a large scale, and a reasonable term of time for experience, there can be nothing discovered of more importance to the construction of durable foundations either under fresh waters or those of the sea, than those certain minerals that I take the liberty of transmitting to you herewith. These will be found in two stone pots to your address. One of them contains a powder which I conceive to possess the properties of Puzzolana, or the cellular basalt of Dr. Rees; the other is a lime produced from certain concretions of lime, clay and other matters found on the banks of York River, near to the town of Little York. These jagged and very various irregular sized (apparently) rocks seem to have been formed a little below the adjacent land, and to have tumbled from them as the washings of the tides have worn them down; for many fragments or distinct masses are seen pendant from their beds. The quantity is very extensive, and from some parts of the same banks the vertebra and other bones of some huge land or sea animal are found to have been dislodged likewise. This lime-rock, upon being calcined, falls to an impalpable powder. It does not slack like other limes; on the contrary, by the application of water as in slacking other limes, the powder forms itself into a mass and coagulates by lying; and when made into a paste, forming a plate of it, suspecting it to dry, it assumes a stony or hard appearance, which being immersed in water before it is quite dry too, it does not dissolve like paste made of other lime.

One of those pots aforesaid (the other) contains a mixture of this lime powder and the powdered basalt, in the proportion of lime two, basalt three, which, from my experiments, seems to be the most perfect for terias mortar. They are to be reduced to a plastic state by adding the smallest quantity of water possible, and that by little and little, to aid the beating in rendering it tough. Observe this rule—the more beating and the less water, the firmer the mortar. Hence you will perceive, sir, that my researches have been to the best chemical authorities, as far as my simple capacity has enabled me to understand from Dr. Rees and some others upon this subject. My acquirements and ability to investigate and to understand are solely from exercising my practical knowledge and limited powers of mind; whilst I would most respectfully solicit your enlightened aid to mature my purpose and to stamp a character upon my inventions.

Two bricks were cemented on the first of this month with a mortar far less perfect than the above, and instantly (while the mortar was soft) they were placed in a basin of water, where they have remained ever since. The cement grows harder with time, as is very perceptible, and from the crust that is evidently forming on the surface I am expecting a crop of Stalactites.

You will readily perceive, sir, that if the invention shall prove effectual, I am fairly entitled to a reasonable compensation, to be secured by a patent or otherwise, and that your kind assistance in the promotion of my object would be gratifying in an eminent degree. In conclusion, sir, I pray you would have the goodness to favor me with a reply, for my apprehensions of having trespassed too far on your benignity can only be relieved by your favorable reception of this appeal to your liberality, and by such orders for a supply of the crude materials as you shall be pleased to give your most respectful and humble servant. D. M. Randolph.

Concerning the writer of this letter, we know merely that his name was David Meade Randolph, and that he lived in tidewater Virginia. The letter, regard being paid to both form and substance, would seem to indicate that he was an Englishman by birth, and of comparatively recent arrival in the United States. Dr. Mitchill, contrary to his usual practice, does not seem to have published any discussion of the subject, and the later history of Randolph and his cement cannot be traced. The entire matter seems to have passed from mind, even in Virginia, for the Shepherdstown plant erected about 1829, was hailed there as inaugurating a new industry for the state.

The materials which Randolph used can be identified with some certainty. His principal product was made by calcining nodules of very clayey (and probably very ferriferous) limestone, which occur at many points in the so-called "marl" deposits of the coastal plain. His other raw material he describes as resembling "cellular basalt," and as possessing the properties of *terias* (i. e., trass). This identification, to judge from the locality, was probably incorrect; and it seems fair to assume that it was really a clayey iron ore, which is also common in this region.

*U. S. Geological Survey, Washington, D. C.

THE STEEL-PILE CAISSONS for the foundations of the new Brevoort Hotel, Chicago, which were mentioned in the article on "Steel Sheet Piling" in our issue of Nov. 23, were the cause of some very unusual foundation work. The intention was to cap the concrete pier (formed by filling the caisson with concrete) with a double grillage of I-beams supporting the column shoe. When the clay was excavated from the interior, however, it was found that none of the caissons were vertical. The reason for this, as we are informed, is that insufficient care was given to the driving of the piles, while the pile driver was not adapted to the work and was so flexible that the leads were not kept vertical. The bottoms of the 30-ft. caissons (whose tops were 12 ft. below the street level) were in firm clay or hardpan, and the shafts were therefore carried down vertically about 35 ft. more, using wooden stave lining in 5-ft. lengths. The shaft was then filled with concrete to the level of the bottom of the steel lining, and capped with an I-beam grillage. Upon this was erected a vertical cast-iron column (inside the inclined shaft) to carry the grillage and the shoe for the structural steel column of the building. This was about 30 ft. high, built in three sections, and after being put in place the shaft was filled

with concrete in which the cast-iron column is embedded. The column load of the building, however, is not carried by this inclined pier of concrete, but is transmitted by the cast-iron column to the vertical pier of concrete below.

A WATER SUPPLY FROM A HOT SPRING is proposed for Socorro, N. Mex. We are informed by the engineer for the project that the spring is at such an elevation that, as planned, it will afford 288 ft. head for a hydro-electric plant, and then the water from it will be utilized for water-works purposes. The Socorro Water & Light Co. has been incorporated to carry out the project; has organized with Mr. G. A. Collins and Frank Owen, both of Santa Fe, as chief engineer and general manager, respectively; and has made preliminary surveys. So far as we know the only other instance of a general hot water supply system in the United States is at Boise City, Ida. At that place, according to "The Manual of American Water-Works" for 1897, the Artesian Hot & Cold Water Co. furnishes both hot and cold water. The hot water (170° F.) is piped through the principal streets and sold for both heating and other domestic purposes. In addition the company owns and operates hot baths, located near its hot water wells. The water at Socorro has a temperature of 87° F.

THE RAILWAYS OF SOUTH AUSTRALIA, owned and operated by the colonial government, have a total length of 2,036 miles (of which 11 miles are double track), besides 208 miles of siding. Of this total 498 miles (including the double track) are of 5 ft. 3 ins. gage, while 1,528 miles are of 3 ft. 6 ins. gage, and 9 miles have the mixed gage. The capital cost has averaged $38,915 per mile. The total earnings for the year ending June 30, 1905, were $6,366,605, of which the operating expenses were $3,683,955, or 57.86%. The colony has a population of about 370,000, exclusive of natives (who presumably do not travel or furnish traffic to any extent); there are thus 212 persons per mile of line, and 4.72 miles of line per 1,000 head of population. The railway equipment comprises 333 locomotives, 432 cars in passenger service and 6,361 cars in freight and railway service. Some steam motor cars for light traffic on branch lines are also being introduced. There is a scarcity of tie timber in the colony, and it has been suggested that a systematic plan of planting suitable forest trees should be taken up. Mr. Alan G. Pendleton is Railway Commissioner; Mr. Alex. B. Moncrieff, Engineer-in-Chief, and Mr. T. Roberts, Chief Mechanical Engineer.

PERSONALS.

Mr. R. C. Sandlass has resigned his position as corresponding secretary for the Technological Society of Baltimore, Md.

Mr. W. H. Gatchell has been appointed Superintendent of the Nashville Division of the Southern Ry., with offices at Nashville, Tenn.

Mr. L. H. Wheaton has been appointed Chief Engineer of the Halifax & Southwestern Ry., with headquarters at Bridgewater, N. S., vice Mr. T. H. White, assigned to other duties.

Mr. F. R. Layng has been appointed Engineer of Bridges, in charge of Bridges and Buildings of the Bessemer & Lake Erie R. R. His headquarters will be at Greenville, Pa.

Mr. Henry N. Rockwell, of Yonkers, N. Y., has been appointed a member of the New York State Railroad Commission. Mr. Rockwell is a conductor in the employ of the New York Central & Hudson River R. R.

Mr. John C. Bentley has resigned his position with the New York, New Haven & Hartford R. R., to accept a position as Assistant Engineer, New York, Ontario & Western Ry., with headquarters at Middletown, N. Y.

Mr. F. D. Chase, Chief Draftsman, Engineering Department of the Minneapolis & St. Louis R. R. and Iowa Central Ry., has accepted a position as Engineer with the Leonard-Martin Construction Co., 802 Monadnock Block, Chicago.

Mr. H. S. Geismer has resigned his position as Division Engineer, Tennessee Coal & Iron R. R., to accept a position with the Republic Iron & Steel Co., as Chief Engineer, with offices in Birmingham, Ala. Mr. Geismer is a graduate of the University of Michigan, Class of '97.

Mr. W. T. Turner, Topographer in U. S. Geological Survey, has been granted a furlough by U. S. Government in order that he might act as assistant to Mr. John W. Wade, State Engineer of Montana, to make plane-table surveys of irrigated land in the Sun River Basin.

Mr. W. R. Armstrong, M. Am. Soc. C. E., resigned his position as Superintendent of Construction, Kansas City, Mexico & Orient R. R., at Chihuahua, Mexico, Nov. 20, to accept a position as Assistant Engineer under Wm. Ashton, Chief Engineer, Oregon Short Line R. R., with headquarters at Salt Lake City, Utah.

Mr. Charles A. Olson, who for several years has been Superintendent of the flanged fitting department of Crane Co., Chicago, has been promoted to the newly created position of General Superintendent of that company. Mr. Olson was formerly Superintendent of the St. Petersburg, Russia, plant of the Societe Anonyme Westinghouse.

Major John Biddle, Corps of Engineers, U. S. Army, whose detail of four years as Engineer-Commissioner of the District of Columbia has expired, will be retained in that position by the President in order that he may complete the work that he has undertaken, most of which is well under way. This includes the new union passenger station, the completion of the sewerage system, and the Connecticut Avenue bridge.

Mr. E. H. Coapman has been appointed Assistant General Superintendent, Eastern District, of the Southern Ry., with headquarters at Greensboro, N. C., vice Mr. H. Baker, resigned. Mr. W. S. Andrews succeeds Mr. Coapman as Superintendent of the Danville Division, Eastern District, of the Southern Ry., and Mr. C. S. Lake has been promoted to the position of Superintendent of the Washington Division, to succeed Mr. Andrews.

Mr. F. A. Molitor, M. Am. Soc. C. E., has been selected by the Bureau of Insular Affairs as the Government's Supervising Engineer for the lines of railways to be built in the Philippines. Mr. Molitor is Chief Engineer of the Midland Valley Ry., and formerly was Chief Engineer and builder of the Choctaw, Oklahoma & Gulf R. R. He takes the position originally intended for Mr. John F. Stevens, who was made Chief Engineer for the Isthmian Canal.

Mr. G. H. Stapp has been appointed Superintendent of the Joplin Division, Missouri Pacific Ry., with headquarters at Nevada, Mo., succeeding Mr. J. M. Walsh, who has been appointed Superintendent of the Northern Kansas Division, with office at Atchison, Kan. Mr. Horace Baker has been appointed General Superintendent of the Southern District, with headquarters at Little Rock, Ark., vice Mr. W. T. Tyler, resigned. Mr. J. Russell has been appointed Superintendent of the Omaha Division, with office at Omaha, Neb., and Mr. G. W. Inge has been appointed Assistant Superintendent of that division. The latter will have his headquarters at Kansas City, Kan.

Obituary.

John B. Morford, Assistant General Manager of the Michigan Central R. R., died Nov. 29 at his home in St. Thomas, Ont., of pneumonia.

James B. Oliver, President of the Oliver & Snyder Steel & Coke Co., and the Oliver Iron & Steel Co., died Nov. 28 at his home in Shields, Pa. He was 61 years old.

Capt. W. M. Rees, Assistant U. S. Engineer, with headquarters at Memphis, Tenn., died Dec. 4, of injuries received on a Government dredge-boat several days ago.

Charles A. Thompson, founder of the Thompson-Bonney Co., electrical contractors of 45 York St., Brooklyn, N. Y., died Nov. 28, at the Brooklyn Hospital of congestion of the brain. He was 44 years old.

William H. Edgar, founder and President of the Dearborn Drug & Chemical Co., of Chicago, and who had made special studies and investigations of the treatment of water for boiler and industrial purposes, died at Hot Springs, Ark., Nov. 26. He was born in Chicago in 1865, and was elected a member of the Western Society of Engineers in 1902.

Samuel Lawrence Minot, at one time Chief Engineer of the Boston & Providence R. R., died Nov. 28 at his home in Malden, Mass. Mr. Minot was born in Belgrade, Me., in 1826. He was graduated from Phillips Academy, Andover, and afterward studied in the office of Thomas Doane. His professional career was as follows: Assistant Engineer of the Charles River R. R. (now Woonsocket Division, New York & New England R. R.); Engineer in Charge of the location and construction of the Newport extension of the Old Colony & Fall River R. R.; Chief Engineer of the Boston & Providence R. R.; Engineer of Construction, Providence Division of the Old Colony R. R.; completed designs accepted by the Railroad Commissioners for elevation of the tracks of the Providence Division to abolish grade crossings in Boston. He was also expert engineer for the Boston & Lowell R. R. under its lease to the Boston & Maine. He was formerly a member of the Boston Society of Civil Engineers and of the American Society of Civil Engineers.

ENGINEERING SOCIETIES.

COMING MEETINGS.

ENGINEERING ASSOCIATION OF THE SOUTH.
Dec. 15, 16. Annual meeting at Birmingham, Ala. Secy., H. M. Jones, Berry Block, Nashville, Tenn.

AMERICAN ECONOMIC ASSOCIATION.
Dec. 27 to 29. Annual meeting at Baltimore, Md. Secy., F. A. Fetter, Ithaca, N. Y.

MICHIGAN ENGINEERING SOCIETY.
Jan. 9, 10, 1906. Annual convention at Lansing, Mich. Secy., F. Hodgman, Climax, Mich.

NATIONAL ASSOCIATION OF CEMENT USERS.
Jan. 9 to 12, 1906. Annual convention at Milwaukee, Wis. Secy., C. C. Brown, Commercial Club Bldg., Indianapolis, Ind.

NEW ENGLAND WATER-WORKS ASSOCIATION.
Jan. 10. Annual meeting. Secy., Willard Kent, Narragansett Pier, R. I.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 17. Annual meeting at New York, N. Y. Secy., Charles Warren Hunt, 220 West 57th St., New York, N. Y.

INDIANA ENGINEERING SOCIETY.
Jan. 18, 19, 20. Annual meeting at Indianapolis, Ind. Secy., C. C. Brown, Commercial Club Building, Indianapolis, Ind.

ILLINOIS SOCIETY OF ENGINEERS AND SURVEYORS.
Jan. 17, 18, 19. Annual meeting at Rockford, Ill. Secy., E. E. R. Tratman, 1636 Monadnock Block, Chicago.

NORTHWESTERN CEMENT PRODUCTS ASSOCIATION.
Jan. 17, 18, 19. Annual convention at Minneapolis, Minn. Secy., F. H. Chapin, 10 N. Third St., Minneapolis, Minn.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Feb. 21. Annual meeting at South Bethlehem, Pa. Secy., R. W. Raymond, 99 John St., New York, N. Y.

NEW ENGLAND WATER-WORKS ASSOCIATION.—At the December meeting of the association, 2 p. m., Dec. 13, Mr. A. O. Doane, of Boston, is scheduled to read a paper entitled "Water Pressure Regulators." The Committee on Private Fire Services will submit a report. A general discussion of Electrolysis is also on the program. As usual, the meeting will be held at the association's headquarters, Tremont Temple, Boston.

BROOKLYN ENGINEERS' CLUB.—The annual dinner, at which also the election of officers will take place, is to be held at the Oxford Club, 109 Lafayette Ave., Brooklyn, N. Y., at 9 p. m. of Dec. 14. The official nominees for officers are: For President to succeed E. S. Buck, W. S. Tuttle; for Secretary, Jos. Strachan, to succeed himself; for Treasurer, John Middleton, to succeed himself; for Directors, L. J. Carmalt and K. L. Martin.

INDIANA MUNICIPAL LEAGUE.—Municipal franchises and municipal ownership of public utilities were the principal subjects discussed at the 14th annual meeting of the league, held at Indianapolis on Nov. 15. The discussion was opened by an address on "Municipal Franchises," by Finley P. Mount, City Attorney, of Crawfordsville. Mr. Mount said his experience is that the success or non-success of municipal ownership of public utilities is largely a matter of bookkeeping; that it is an easy matter to render reports or trial balances, either for or against municipal ownership; and that it is absolutely impossible to divorce their management from politics. He gave a history of the trials and tribulations encountered by the electric light plant at Crawfordsville, owned by the city, saying that for fifteen years it was a losing proposition, and while it is now being operated by a bi-partisan board in a paying and satisfactory manner, the experience has been a very expensive one to the city. The water-works in Crawfordsville, operated by a private corporation, give excellent service, which he attributed to the strength of the contract with the water company. No reliance, he said, could be placed in a forfeiture clause in a franchise, especially in a water franchise. The strength of a water franchise lies in the financial penalties that may be assessed and withheld from the next quarterly payment.

In discussion, Mayor Becker, of Hammond, led in an argument in favor of municipal ownership of public utilities excepting when the indebtedness of a city precluded in favor of a grant of franchise to a private corporation in order to get service. He described how the municipal water-works plant of Hammond was saving the city $30,000 a year. He condemned the methods of private corporations to issue bonds on the earning capacity of a plant rather than its actual cost, and said private corporations have been known to control city elections and in such case they are a menace to a municipality. Mayor France of Huntington said that he favored municipal ownership; that private corporations were gaining a stronger and more arrogant hold on the Government, and that it is time to crush this arrogant power. He insisted that party politics should not enter municipal affairs; that the day was coming when it would have no place in municipal affairs. William Fortune, editor of "The Municipal Magazine," said that he favored municipal ownership of water plants, that there is no more reason why a municipal government should be a failure in the management of public utilities than in the management of other affairs. He said the better plan would be to allow the people to vote on an important franchise before it was granted to a private corporation.

A paper was next read on "Municipal Ownership of Water and Light Plants," by H. C. Ayers, Mayor of Portland. Mr. Ayers' paper covered much the same ground and reasoning assumed by Mr. Mount. The delegates participating in the discussion were about equally divided in favor of municipal and of private ownership and control of public utilities.

The following officers were elected for the ensuing year: President, Fred. J. France, Mayor of Huntington. Vice-Presidents, B. R. Russell, ex-Mayor of Crawfordsville, and George R. Carr, Mayor of Greenfield; Secretary, John F. Taggart, Clerk of Richmond. The next meeting will be held in Marion, June 15, 1906.